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Relationship between support structure forces and anchor bolt stresses

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RELATIONSHIP BETWEEN
SUPPORT STRUCTURE FORCES
AND ANCHOR BOLT STRESSES

by

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Abstract

Experimental and analytical research was performed to determine the relationship between axial anchor bolt stresses and support structure forces for cantilevered sign, signal, and luminaire support structures. Predictions of this relationship were made using the flexure equation and the moment of inertia of the bolt group. Static tests were conducted on four anchor bolt assemblies to study the relationship between the structure fores and bolt stresses. The impact of difficult to quantify parameters such as, misalignment, torsion, base plate thickness, exposed bolt length, looseness of nuts, and pretension in bolts, were investigated. The results of these static tests were in good agreement with the analytical predictions. Full-scale fatigue tests were also conducted on the anchor bolt assemblies at a 20 ksi stress range. The failures were consistent with data obtained from the uniaxial fatigue tests of individual anchor bolts.

Chapter One

Introduction

1.1 Statement of Research

Cantilevered sign, signal, and luminaire support structures are used extensively on major interstate highways and at local intersections for the purposes of traffic control and roadway illumination. Over the years, the span of these cantilevers has been increasing as they are utilized on roads with more lanes and as the setback distance of the column from the roadway has been increased for safety reasons. Today, it is not unusual for spans to be more than 40 feet in length. Although a single vertical support, as opposed to two supports for traditional overhead structures, increases motorist safety by minimizing the probability of vehicle collision, the flexibility of the cantilevered structures is significantly increased.

This flexibility, combined with a low mass, results in resonant frequencies of about 1 Hz. The damping of these structures is also extremely low, usually less than one percent of the critical damping. These conditions make cantilevered support structures particularly susceptible to large-amplitude vibration and fatigue cracking due to wind-loading.

Traditionally, these support structures are designed in accordance with the AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals with minimal performance problems. However, through a state department of transportation survey conducted as a part of previous research on this project, some states indicated problems with wind-induced vibration of cantilevered support structures [1] While several states reported horizontal mast-arm vibration amplitudes in excess of 25 in under steady-state winds with velocities in the range of 11 mph to 33 mph, most of the vibrations have been observed to occur in the plane of the structure (i.e. vertical-plane vibrations of the horizontal mast arm) in a direction normal to the direction of wind flow [1]

These large-amplitude vibrations that have been observed to induce stress ranges in the structure. Sometimes these stresses are relatively small, creating merely a serviceability problem. However, in many cases, the magnitudes of these stress ranges initiate fatigue cracks in critical connection details which eventually could cause the collapse of the structure. The most susceptible connection details to be reported include the mast-arm-to-column connection, column-to-base connection, and the anchor bolts.

In many of the observed fatigue failures, cracking has initiated in the anchor bolts where the stress ranges result from vertical in-plane motion caused by transverse motion in any direction or from twisting of the column on the fixed base. The forces from these movements are transferred into the bolts through the base plate. According to the survey mentioned above, most of the states surveyed cited a total of 85 occurrences of fatigue damage on cantilevered support structures with truss arms. States (such as Connecticut, Michigan, West Virginia, and Georgia) reported that 42 of these fatigue failures have occurred in the anchor bolts of these structures. Among the factors that contribute to fatigue cracking are prying from the end plate thickness and geometry, the bolt pattern and installation conditions and the magnitude of the stress range. The majority of the failures were attributed to a lack of pretension in the bolt, while the rest occurred in bolts with unknown installation conditions. These failures indicate that problems that can occur during the installation process affect the uncertainty in knowing not only the constant amplitude fatigue limit (CAFL), but also the distribution of stresses in the anchor bolt assembly.

1.2 Objective

This research investigates the effect of common problems in the design of support structures and the relationship between support structure forces and anchor bolt stresses. Common problems include misalignment, pretension in the bolts, the exposed bolt length between the leveling nut and the concrete base, and the base plate thickness. Although the theoretical relationship between the anchor bolt stresses and support structure forces is relatively straight forward, the above problems factor significantly into the design of the anchorage assembly by altering the distribution of stresses within the bolts. As part of this study, the adequacy of the simple equilibrium rigid body model used to transform moments and shears acting on the base connection into bolt forces will be assessed.

1.3 Summary of Approach

Full-scale static tests were conducted on a specimen with an eight-bolt anchor assembly to determine the relationship between support structure forces and anchor bolt stresses. The effects of bolt misalignment, exposed bolt length, plate thickness, pretension in the bolts, and nut looseness were evaluated. The specimen is further described in Section 2.3.

Full-scale fatigue tests were also performed to verify the results of individual anchor bolt fatigue tests conducted through previous research to determine lower-bound estimates of the fatigue strength of axially-loaded, snug- and fully-tightened anchor bolts in the regimes of finite and infinite life [2].

1.4 Thesis Organization

Chapter Two provides background information pertaining to the theoretical relationship between support structure forces and anchor bolt stresses. Chapter Three summarizes the experimental tests, both static and fatigue. A description of the methods used to analyze the data is also presented in this chapter.

Chapter Four presents the conclusions and synthesizes the results of this research in a series of guidelines for design. In addition, recommendations for future research are included.

Chapter Two

Relating Support Structure Forces to Bolt Stresses

2.1 Introduction

Variations in numerous site-specific parameters, such as misalignment, exposed bolt length, and tightness of the nuts, can significantly alter the distribution of stress between bolts within a bolt group as well as within any single bolt. Theoretically, misalignment of the entire anchor bolt group (a fixture typically keeps them mutually parallel) relative to the vertical axis tends to influence the stress distribution within the anchorage assembly in two distinct ways. First, misalignment produces eccentric axial forces in each of the bolts. These eccentric forces induce bending moments in each bolt which increases the magnitude of the extreme fiber stress. Second, misalignment tends to increase prying forces in the bolts. These prying forces generate localized bending moments in the bolts between the top nuts and the leveling nuts. The localized bending also increases the magnitude of the stress in the bolt threads adjacent to the baseplate connection. The installation of bolts with bevelled washers has been considered to reduce localized bending due to non-uniform bearing of the nut against the base plate, thereby reducing the stresses.

The exposed bolt length between the bottom of the leveling nut and the top of the concrete foundation influences the stiffness of the anchorage assembly.

Increasing the exposed length increases bending moments (and bolt stresses) generated from shear forces applied through the column base plate while reducing the torsional stiffness of the anchorage assembly.

Finally, the inability of one or more loose bolts to effectively carry loads applied to the structure results in a redistribution of anchor-bolt forces. This condition is experienced after fracture of a bolt or if a bolt was not properly tightened during construction or loosened during service. If all the bolts are properly tightened, a pretension occurs between the nuts reducing the actual stress range in the section of bolt between the nuts. All these conditions result in uncertainty in knowing the distribution of stresses in the anchor bolt assembly.

2.2 Load Conditions

The main loads acting on most cantilevered support structures are gravity and live loads from wind or truck induced gusts. Forces due to gravity loads include the dead weight of the structure plus the weight of any ice or snow that may cling to the structure. The latter, of course, depends on the time of the year as well as the geographical area in which the structure is located. Determination of the magnitude and effect of these static loads is relatively simple and straight-forward.

Wind loads, however, are considered to be dynamic in nature, requiring a more complex analysis to determine the theoretical effect on structures. Four types of wind loading have the ability to affect the vibration of cantilevered support structures: galloping, vortex shedding, natural wind gusts, and truck-induced wind gusts. Both galloping and vortex-shedding are classified as aeroelastic phenomena caused by a coupling between the aerodynamic forces which act on a structure and the structural vibrations [3]. Primarily, galloping-induced oscillations occur in flexible, lightly-damped structures with non-symmetrical cross-sections and are caused by forces which act on a structural element as it is subjected to periodic variations in the angle of attack of the wind flow. Since the majority of cantilevered support structures consist of members of circular cross-section, galloping-induced vibrations occur due to the attachments rather than the structure itself.

Vortex shedding results from vortices which form alternately on opposite sides of the structure during steady, uniform flows and produces resonant oscillations in a plane normal to the direction of flow. Due to the geometry of and placement of attachments on cantilevered support structures, vortex shedding appears to affect the members of the structures rather than the sign or signal attachments. However, the dimensions of the structural members must be very large (e.g. greater than 35 in diameter) in order to be susceptible to the vortex shedding phenomenon. Also, tapered sections have been found to be even less susceptible than prismatic sections.

All structures are susceptible to natural wind gusts and truck-induced wind gusts, however, the level of response depends on the geometry of the structure, and the geometry and placement of the attachments on the structure. Natural wind gusts arise from the inherent fluctuations in the velocity and direction of air flow which induce fluctuating pressures on the various structural components of a wind-loaded structure, resulting in vibration in that structure. Due to the low damping of cantilevered support structures, the response to wind gusts is significant with the susceptibility of excessive deflections and fatigue cracking. Passing of trucks beneath cantilevered support structures induce horizontal gust loads on the frontal area and vertical gust loads on the underside of the signs and members, which result in torsional and bending moments in the column supports. The vertical loads on the underside of the parts are believed to be the most critical. As expected, it is found that the larger the horizontally projected area of the attachment is, the more susceptible the structure is to truck-induced wind gust loading.

From prior research that considered the previously mentioned types of wind loadings, a maximum wind pressure on a cantilevered support structure was estimated based on the different response due to the different loadings [4]. The static equivalent of this maximum pressure was then calculated and applied to the model to determine the maximum strain created in the anchor bolts. The response of the structure to the dead load was calculated, and it was determined that the effect of wind loading is more significant than that of the dead load. Therefore, the response due to dead load will not be taken into account for the static tests. However, for the fatigue tests, the dead load influences the fatigue process and the final size of the fracture; and, therefore, the dead load will be taken into account when calculating the maximum and minimum stresses in and the magnitude of the stress range used during the full-scale fatigue tests.

2.3 Test Fixture Design

Two test specimens were used, each consisting of two components: a cast-in-place foundation with embedded galvanized anchor bolts, and a fabricated steel column stub with base plate. These parts were exchangeable, creating four separate assemblies described in the test matrix in Table 2.1. The anchor bolt size and pattern used were selected as a representative of small to mid-sized cantilevered sign support structures with mast arm spans from 10 ft to 30 ft.

The two concrete foundations were designed and detailed conservatively with eight 1.5 in diameter anchor bolts, placed in a 21 in diameter circular pattern, to provide a shear and moment capacity in excess of the predicted anchor bolt group ultimate strength. The anchor bolts were fabricated to AASHTO M314-90 "Standard Specification for Steel Anchor Bolts" (Grade 55) with a 6UNC (rolled) thread series. One concrete foundation was cast with a minimum possible misalignment in the anchor bolts, since it is recognized that some level of misalignment is inevitable in the construction of these anchorages. The second foundation was cast with all eight bolts racked at the maximum allowable misalignment angle of 1:20 from the vertical. At the time of casting the misaligned foundation, an angle of 1:20 was the maximum misalignment allowed in the Michigan DoT anchor bolt special provisions. This provision is the only one known to qualitatively define a maximum allowable anchor bolt misalignment. However, since then, a revision of the Michigan DoT provision allows only an angle of 1:40 from the vertical as the maximum allowable misalignment.

The two 16 in diameter by 10 ft long steel column stubs were fabricated with a socket-type connection between the column and base plate by a national supplier of cantilevered support structures. Both specimens were detailed identically with the exception of the base plate thickness. One stub was made with a 1.5 in thick base plate, and one with a 1 in thick base plate. Chosen so that its thickness is greater than or equal to the bolt diameter, the 1.5 in thick base plate represents typical support structure base plates and was tested with Assemblies 1 and 3. Assemblies 2 and 4 used a column-stub specimen with a 1 in thick base plate, which was intentionally undersized to investigate the possible effect of base plate stiffness on load distribution. Figure 2.1 depicts the setup as used during a test on Assembly 1 with torsion.

A reusable test fixture with a cantilever arm was designed to easily attach to the top of both column stubs and to allow static tests to be performed at four different Moment:Torsion:Shear (M:T:V) ratios by applying loads to the test fixture at four different locations. The elevation and plan of the entire setup are shown in Figures 2.2(a) and (b). These four M:T:V ratios as well as the method of selection will be described in Section 3.5.1. Two loadings included torsion while the other two did not.

2.4 Analysis Method

In order to facilitate the comparison of the analysis and experimental values, all predictions were calculated using a unit load applied to the specimen. Figure 2.3 shows the three types of loads (moment, shear, and torsion) that comprise the force-couple system that acts on the bolt group. The couple vector M causes the members to bend about the neutral axis and produces normal or axial stresses in the bolt section. On the other hand, the twisting couple T and the shearing force V produce shearing stresses in the bolt section, creating bending of the bolt.

Predicted axial bolt stresses were calculated from the flexure formula (M*c/I) where I is the moment of inertia of the entire bolt group (679 in^4) and c is the perpendicular distance from the neutral axis to the bolt (10.5 in, 7.42 in or 0 in). Shearing stresses were determined using F/A, where A is the cross-sectional area of each bolt (1.538 in^2). The maximum stress was then estimated by taking the square root of the sum of the squares of the axial stress and the total shearing stress. However, for the bolts located on the neutral axis, the total stress was simply the sum of the two shearing stresses, since the bending stress is zero. The simplistic nature of this calculation was also due to the fact that the torsion and shear act in the same planar direction, whereas on the other six bolts, the torsion and shear are either 45 or 90 degrees apart. Complicating the calculation, however, was the placement of the bolt in reference to the X-X axis. Depending upon which side of the X-X axis the bolt was on, the shearing stresses changed signs altering the sign of the total stress in the bolt. As can be seen in Figure 2.3, on one side of the X-X axis, the stresses due to torsion and shear have the same sign, while on the opposite side of the X-X axis, the stresses are opposite in sign.

The bending moment induced by the torsion and shear forces is also dependent upon the exposed bolt length, since the moment is about the intersection of the bolt and the concrete foundation. The greater the length, the higher the moment should be. Using the simple static models of a fixed-fixed beam or a fixed-end cantilever beam, this moment can be predicted. Assuming that the force acts on the bolt at the middle of the base plate, the section can be modeled either as a fixed-fixed beam with the load acting at the middle or as a fixed-end cantilever beam with the load acting at the free end. The length of the beam consists of the exposed length, the width of the leveling nut, and half the base plate thickness. Figure 2.4 depicts the static models used to predict the bending stress in the bolts. The lengths used for each exposed bolt length (1 in and 3 in) are also shown for each model.

The analysis of the model revealed that the moment had the most significant effect of the three types of loading. Between the forces causing the shearing stresses, the torsion load resulted in higher stresses than the shear. The predictions for the bending moment show that, for the 3 in exposed length, the moment is larger than for the 1 in length as expected. In Chapter 3 actual numbers for predicted stresses will be discussed as they are compared to the experimental results.

	Number of Bolts	Bolt Diameter (in)	Bolt Pattern Diameter (in)	Base Plate Thickness (in)	Column Shaft Diameter (in)	Misaligned Concrete Base
Assembly 1	8 (round)	1.5	10.5	1.5	16	N
Assembly 2	8 (round)	1.5	10.5	1.0	16	N
Assembly 3	8 (round)	1.5	10.5	1.5	16	Y
Assembly 4	8 (round)	1.5	10.5	1.0	16	Y

Notes:

1. A total of 6 load cases will be examined for each assembly.

   - Moment:Torsion:Shear ratios of 19.5:0:1, 19.5:5:1, 14.5:0:1, 14.5:10:1

   - Exposed bolt lengths of 1 in, 3 in

   - Each load case will also include loosening two of the nuts sequentially

Table 2.1 Anchor Bolt Group Test Matrix

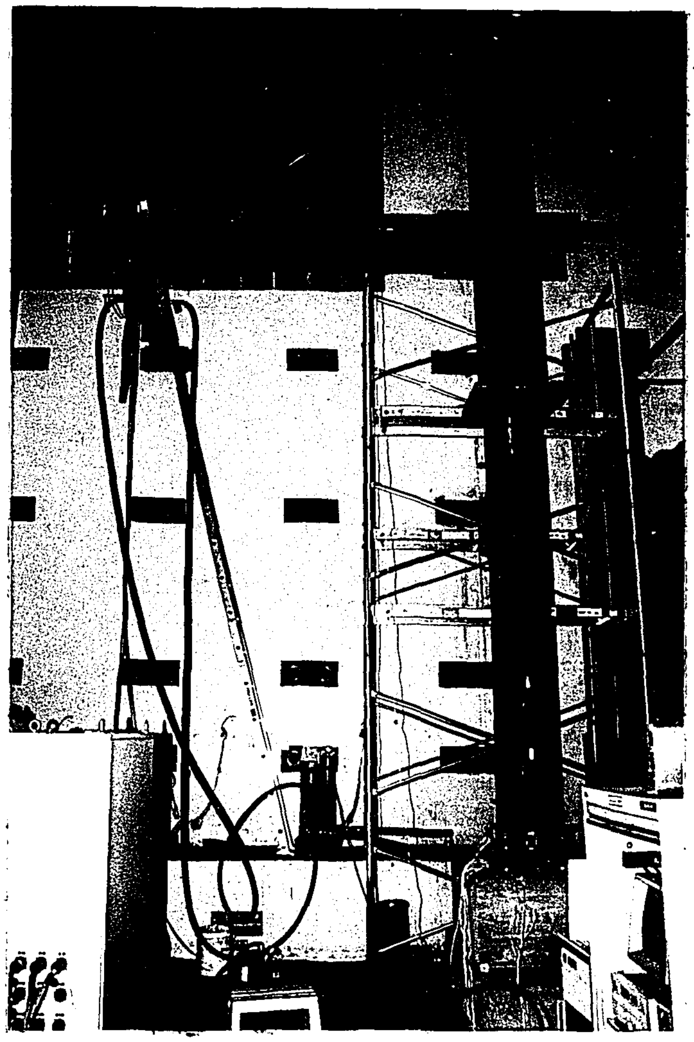


Figure 2.1 Photograph of Overall Test Setup

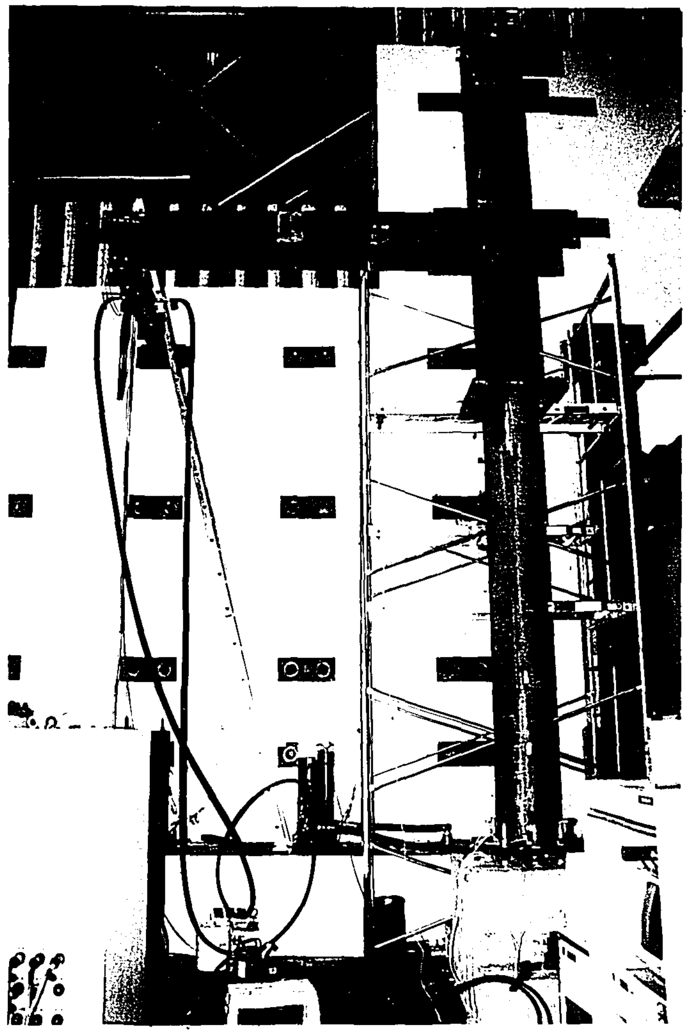


Figure 2.1 Photograph of Overall Test Setup

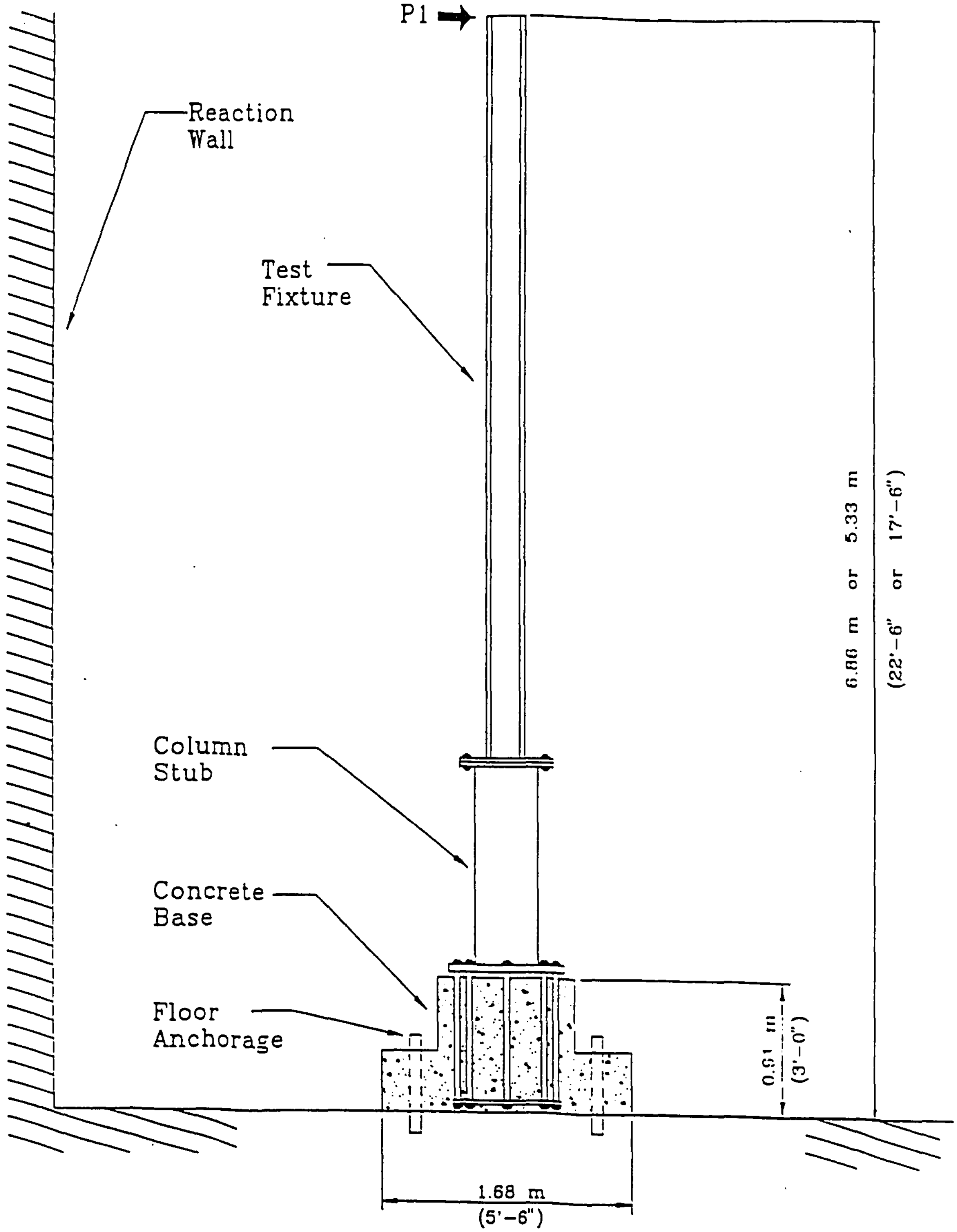


Figure 2.2 (a) Test Setup Elevation

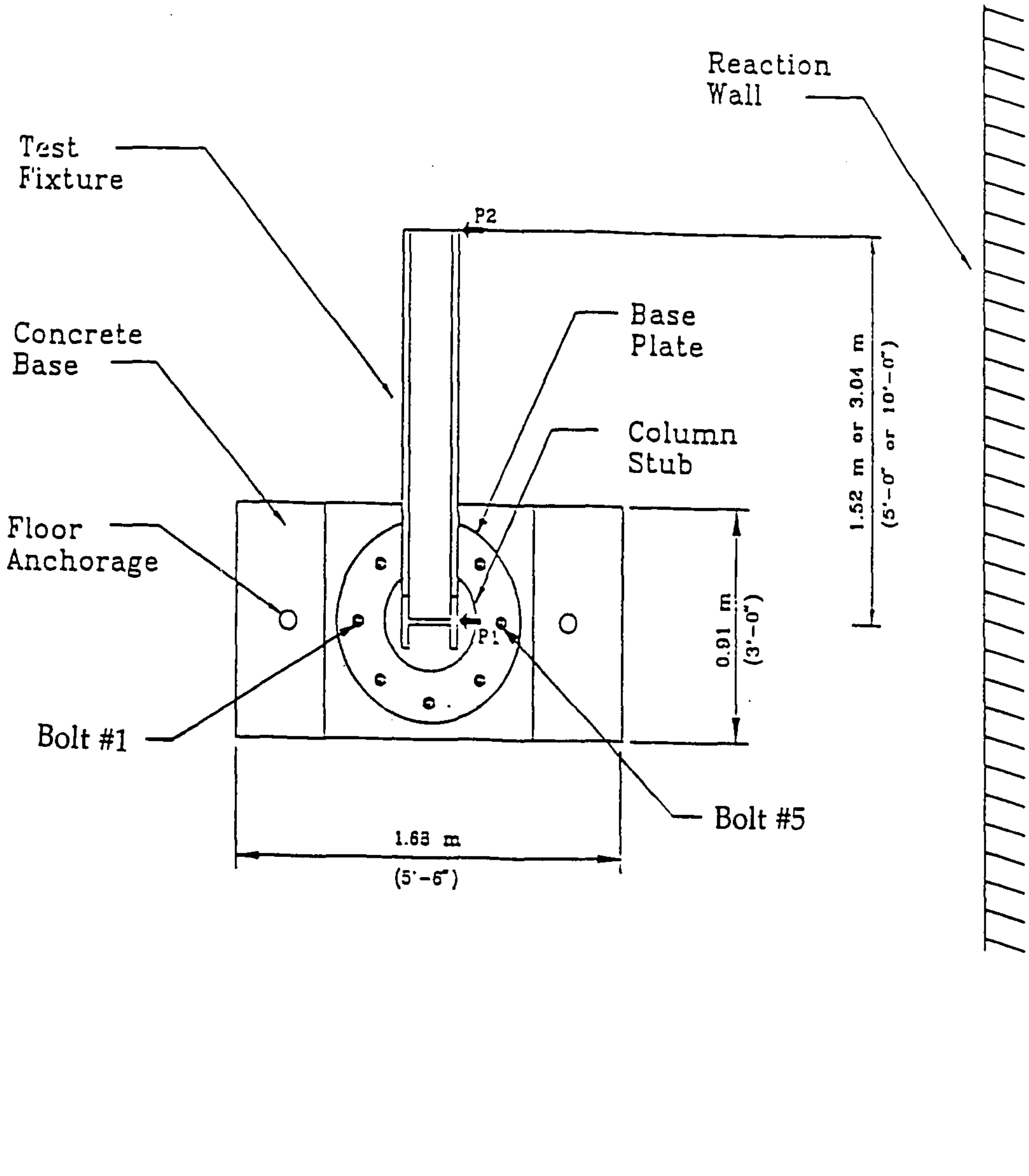
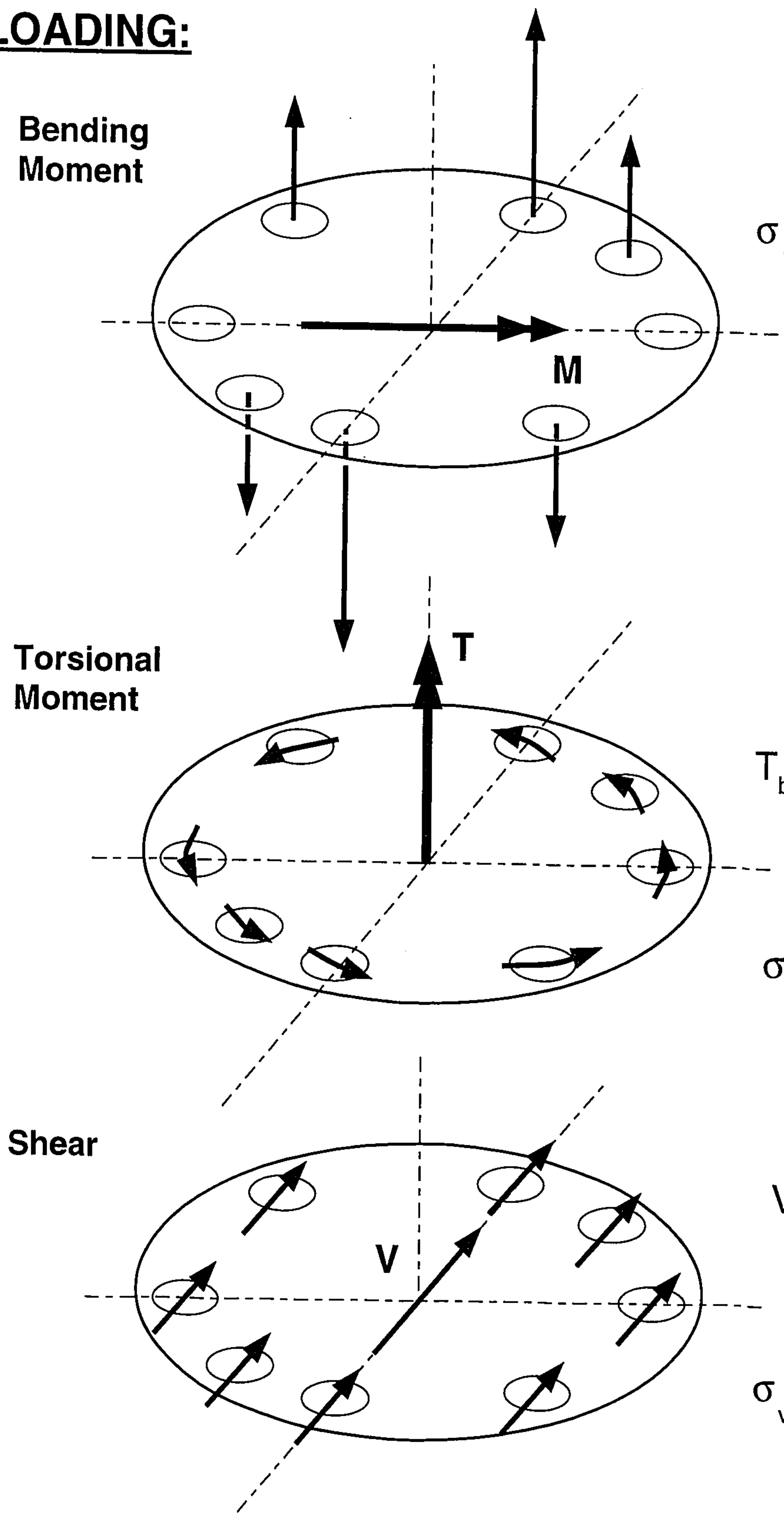


Figure 2.2 (b) Test Setup Plan

LOADING:



$$\sigma_m = \frac{M * c}{I}$$

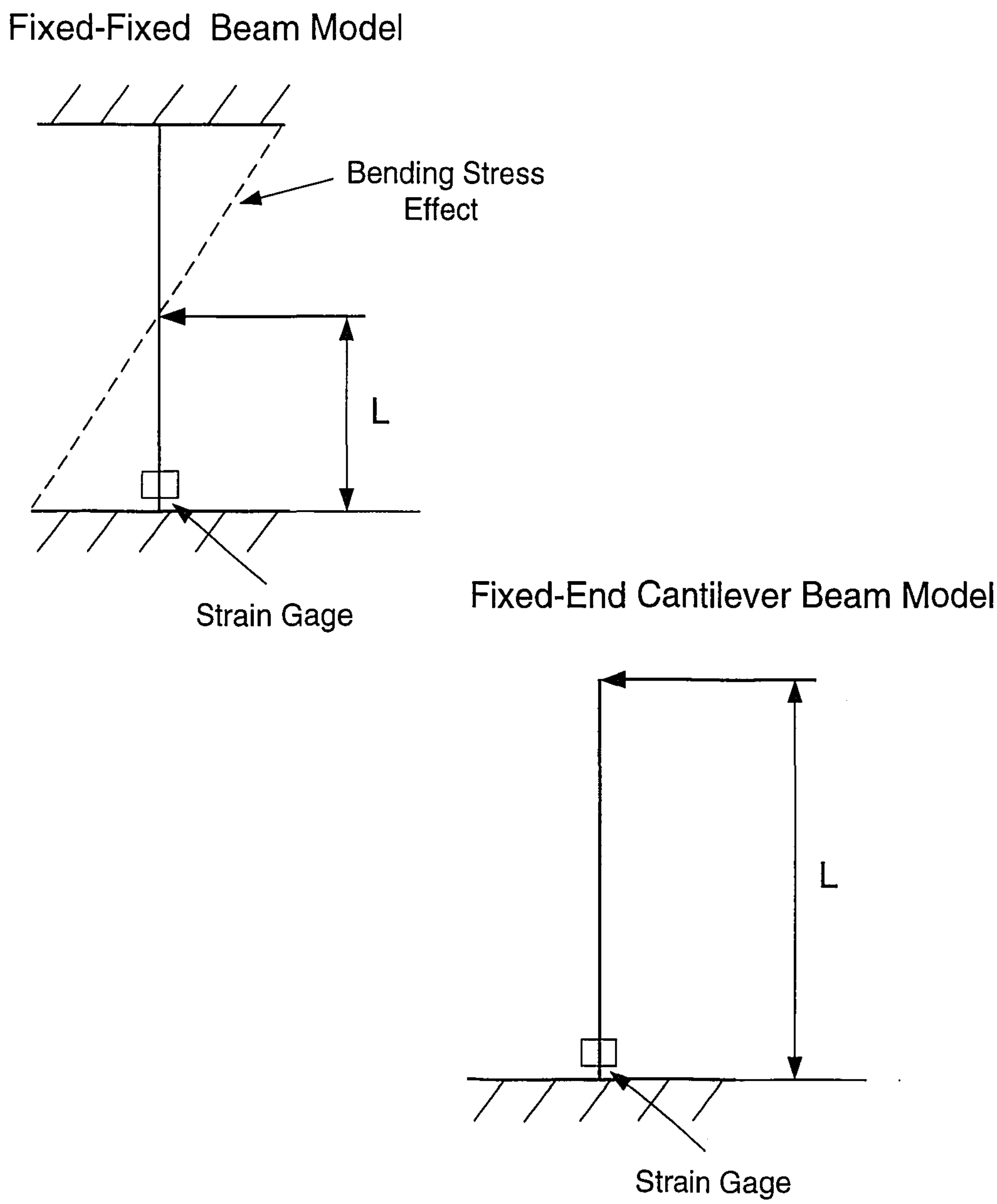
$$T_b = \frac{T}{8 * R}$$

$$\sigma_t = \frac{T_b}{A_b}$$

$$V_b = \frac{V}{8}$$

$$\sigma_v = \frac{V_b}{A_b}$$

Figure 2.3 Force-Couple System Acting on the Anchor Bolt Group



Beam Length (L)

3 in Exposed Length - 5.25 in

1 in Exposed Length - 3.25 in

Figure 2.4 Schematic of Static Models Used to Predict Bending Stresses due to Shearing Forces

Chapter Three

Experimental Results

3.1 Introduction

Although the results of field inspections of cantilevered support structures have indicated that the occurrence of loose and misaligned bolts is relatively common, little or no research has been conducted to study the effects of these parameters on the load distribution in the anchor bolts. Other parameters such as the amount of bolt preload, exposed bolt length and base plate thickness, also may affect the load distribution in the bolts (and therefore the fatigue strength of the bolts). Therefore, since the effects of the above parameters are difficult to quantify analytically, an experimental study was conducted to investigate the distribution of forces in the anchor bolts resulting from forces on the support structure.

In the study both static and fatigue tests were conducted on four assemblies which consist of two concrete foundations and two steel column stubs as described in Section 2.3. During the static tests, the actuator, which applied the load to the specimen, was positioned at four different Moment:Torsion:Shear (M:T:V) ratios: 19.5:0:1, 19.5:5:1, 14.5:0:1, and 14.5:10:1. Through static analysis, the applied forces are predicted to produce two stress components in the bolts. These stresses are axial, caused by the moment couple, and shearing and bending, induced by both the shear force and torsion couple. Using the flexure equation,

$\sigma = M^*c/I$, where I is the moment of inertia of the entire bolt group and c is the perpendicular distance between the neutral axis and the bolt (10.5 in, 7.42 in, and 0 in), the axial stress can be predicted for all eight bolts. Assumptions were then made regarding the effect of site-specific parameters have on these stresses.

Traditionally, a parameter that affects the distribution of stresses in the anchor bolt assembly has also been known to affect the fatigue strength of the anchor bolts. Fatigue tests have been performed on anchor bolts subjected to high and low stress ranges. As a result, the constant amplitude fatigue limit (CAFL) is known for anchor bolts loaded in axial tension. Considering the regime of infinite life, the CAFL corresponding to AASHTO Category D (7 ksi) should be used to design axially-loaded bolts.

The static tests were performed in order to evaluate if the assumptions made were correct and to determine if using the flexure equation is appropriate to predict the distribution of stresses in the bolts. In addition, fatigue tests were conducted on a complete anchor bolt assembly to verify the results of uniaxial fatigue tests on individual anchor bolts.

3.2 Instrumentation

Strain gages were used to measure the amount of deformation in the anchor bolts and the column stub. Each of the anchor bolts in the two concrete foundations was instrumented with four uniaxial strain gages. These strain gages, laid-out at 90 degree intervals around the bolt, were placed on the shank portion of the anchor bolt directly above the concrete foundation. On each of the eight bolts, the gages were oriented with respect to the primary axes (parallel and perpendicular to the direction of the applied load) of the column base rather than radially around the anchor bolt pattern. A close up photograph of the column base and anchor bolt pattern is shown in Figure 3.1.

Strain gages were also placed on the column stub to verify the overturning moment at the column base. Uniaxial strain gages, as well as biaxial rosettes, were positioned at 90 degree intervals around the column stub at 2 in and 4 in above the base plate. These gages were also oriented with respect to the primary axes of the column. The results were consistent with analytical predictions and will be discussed in more detail below. A schematic showing all the different gage orientations and placements is depicted in Figure 3.2.

3.3 Data Acquisition System

All strain gages on the bolts, strain gages on the column stub, and the applied load were continuously monitored by a computer-controlled data acquisition system. To make the collection of data easier, the ATLSS software known as HBMWARE was used in conjunction with the Hottinger Baldwin Messtechnik (HBM) data acquisition system. This software made it possible to view the data during the test, as well as to save the data into a text file that could later be imported into a spreadsheet for easier data analysis. Other features such as two X-Y plots to keep track of the relationship of the load vs strain in the extreme bolts and the ability to record data at specific points rather than at pre-set time intervals aided in the test procedure. Data were saved at discrete load increments on both the loading and unloading portion of the test.

3.4 Construction Sequence

The steel column stubs were attached to the foundations following standard construction practices for ungrouted, double-nut anchor bolt installation. Each anchor bolt in the group was tightened to one-third of a turn past snug, creating pretension in the bolts. The behavior of the bolts during the tightening process was monitored by recording the strain of the bolts.

Tightening to one-third of a turn required the use of a hydraulic wrench and became quite time consuming. However, for static tests, fully tightened anchor bolts may not be necessary to obtain reliable strain gage data. To prove this assumption, the first test was repeated but this time the anchor bolts were tightened using conventional hand-tightening procedures to a rotation of approximately one-sixth of a turn beyond snug. Based on consistent results between this experiment and the experiment with fully-tightened bolts, the remaining tests were conducted with the anchor bolts tightened by hand to as close to one-sixth of a turn as possible.

After casting the concrete foundations, several anchor bolts from the "straight" fixture were calibrated to known tension and compression loads. The purpose of these calibrations was to ensure that the strain gage data was not significantly influenced by the close proximity to the applied load from either the nut or the concrete base. Results of these calibrations coincided with the expected linear relationship between the applied load and average axial bolt strain.

3.5 Static Tests

3.5.1 Loading Procedure

As shown in the test matrix in Table 2.1, each of the four assemblies was tested statically under six load conditions to study the effect of various parameters (i.e. misalignment, exposed length, etc.). For each load condition, the specimen was monotonically loaded in tension and compression to loads slightly below the current static design allowable for anchor bolts (i.e. 0.5 F_y), applied with a 30 kip servo-hydraulically-controlled actuator with a load cell. Through an evaluation of the magnitude of the loads for which cantilevered sign support structures are designed, the four actuator positions were selected based on the fact that the load ratios are sensitive to structural configuration and total sign area. Typically, the M:T:V ratios for small to mid-sized cantilevered sign structures are in the range of 30:2:1 (i.e. for high mast luminaires) to 14:17:1 (for structures with long mast arms). By conducting the tests at load ratios in the range for which actual structures are designed, a more accurate representation of the bending, shear, and axial loads induced in the anchor bolts was achieved.

Two tests were conducted with the actuator positioned at a location which induced a torsion moment and two tests without. In two tests, the actuator was mounted horizontally at a height of 22.5 ft above the lab floor and positioned with and without an eccentricity of 5.0 ft from the vertical axis of the column stub. These two tests investigated the relationship of support-structure forces and anchor-bolt stresses at M:T:V ratios of 19.5:0:1 and 19.5:5:1, respectively. In the other two tests, the actuator was mounted 17.5 ft above the lab floor with and without an eccentricity of 10.0 ft, giving the relationship at M:T:V ratios of 14.5:0:1 and 14.5:10:1, respectively.

Two additional tests were conducted on each assembly with the actuator located in the first two positions (M:T:V ratios of 19.5:0:1 and 19.5:5:1). The first test studied the effect of loose nuts or failed bolts by sequentially loosening and then re-tightening one bolt from the eight bolt group. The bolt selected for loosening was located furthest from the neutral axis of the bolt group. From the results of these tests, the new distribution of stresses in the bolts, especially in the two bolts on either side of the extreme bolt, could be determined after the load was transferred when bolt #1 failed. Finally, one test on each assembly evaluated the effect of variations in the exposed anchor bolt length above the concrete foundation. All tests were first conducted with approximately 1 in of bolt exposed between the top of the concrete foundation and the bottom of the leveling nut. This configuration was selected for testing because it is commonly specified by state DoT's in cantilevered support structure standard drawings. The results are compared to those obtained when approximately 3 in of bolt is exposed above the concrete foundation.

3.5.2 Column Forces to Anchor Bolt Stresses Prediction

As described in Section 2.4, force-to-stress relationship predictions were determined for a unit applied load. Referring to Figure 2.3, stresses due to the moment, shear, and torsion vectors were calculated. The axial stresses due to the moment were the most significant of the three by a large percentage, while the stresses due to the torsion altered the maximum stress more than the stress due to the shear.

Table 3.1 shows the results of these calculations for Assembly #1 (no misalignment and 1.5 in base plate with 1 in exposed bolt length). Since no bending stress should be present in the neutral axis bolts, by comparing bolts #1 or 5 with #3 or 7, the significance of the moment can be seen. For position 1, the stress in bolt #3 due to the shear force is 0.0813 ksi while the combination of axial and shear stress is 3.671 ksi in bolt #1. Also, the results for positions 1 and 3 could be compared to determine the effects of the torsion couple. For example, comparing the predictions for bolt #3, the stress is 0.0813 ksi when the unit load is applied without torsion and 0.545 ksi with torsion.

The combination of the torsion and shear forces with the exposed bolt length also produces a bending moment in the bolt shaft about the point where the bolt meets the concrete foundation. This bending moment was predicted using both a fixed-fixed beam model and a fixed-end cantilever beam model. For the fixed-fixed model, the load is assumed to act at the mid-point of the beam. Whereas, for the cantilever beam, the load acts at the free end. It was assumed that the load acts on the bolt at the mid-point of the base plate, requiring the length used for the predictions to be slightly greater than the exposed length. Represented in Table 3.2 are the results for both models, calculated for both an exposed length of 1 in and 3 in, and with and without torsion. The effect of the base plate thickness on these calculations was also determined since the plate is a large percentage of the length, even though the difference between the two base plate sizes is minimal.

The effects of misalignment of the bolt group as well as the base plate thickness on the predicted stresses were also examined. The original predictions in Table 3.1 were calculated for Assembly #1 (1.5 in base plate with no misalignment). Theoretically, misalignment should increase the stresses in each bolt. The angle of 1:20 produces an eccentric axial stress and a larger cross-sectional area through which the shearing stress acts. Also, since the relationship between the eccentricity and exposed bolt length is linear, the stress should increase linearly. Decreasing the base plate thickness should, theoretically, decrease the stresses since the length from the point of load application decreases. However, it is assumed that, since the difference in base plate thickness used is minimal, there is no effect on the stresses. Prying forces produced by misalignment also tend to increase the stresses, but are difficult to estimate. As a result, the original predicted stresses were compared to the experimental results to determine if the assumptions that no effect occurs due to misalignment of the anchor bolts or a change in base plate thickness is accurate.

3.5.3 Results

3.5.3.1 Anchor Bolts

Raw strain-gage data were examined and, prior to the analysis, data from gages with uncharacteristic readings were deleted. For a few of the bolts, the readings from more than one gage had to be eliminated, leaving the readings from only two gages to analyze, resulting in difficulties in defining an accurate comparison. First the strains from the gages on each bolt were averaged together to obtain an axial strain in each bolt. This averaging helped to reduce the natural error in the measurements. In addition, it eliminates the effect of bending of each bolt, resulting in the stresses in the neutral axis bolt (bolt #3 and #7) to become zero, since only the shearing forces create stresses in these bolts. Then, after the axial effects on the bolts were determined, the raw strain gage data was re-examined to define the effect of bending on the bolts.

Average Axial Tension - Having averaged the strain gage readings per bolt, the strains were analyzed to define the effects of misalignment, torsion, base plate thickness, pretension in the bolts, and looseness of the nuts. As illustrated in the diagram in Figure 3.2, bolts #1 and #5 were located along the axis of loading at a distance of 10.5 in from the neutral axis and, therefore, are the highest stressed bolts in the pattern. A plot of axial strain in each of the eight bolts at incrementally applied loads is presented in Figure 3.3 for the 1 in exposed bolt length on Assembly #1 (i.e. 1.5 in thick base plate on a straight anchor bolt pattern) and the actuator positioned at a M:T:V ratio of 19.5:0:1. In correlation with the results from the calibration test, the relationship between the applied force and the strains in the bolts was linear. As expected, the axial strain in bolts #1 and #5 are the highest of the bolt group and the data are symmetric for applied loads in both tension and compression. Also, the magnitude of axial strain in bolts #2, #4, #6, and #8 is consistently lower than the strain in bolts #1 and #5 by approximately 60 to 80 percent, consistent with the expected geometric relationship. Finally, due to bolts #3 and #7 placement on the neutral axis of bending, measured strains in these components were negligible. Similar results for the relationships between the various individual bolts and the linear load vs strain relationship were obtained for tests conducted with the 1 in base plate, 3 in exposed bolt length, and misaligned anchor bolt pattern.

In order to evaluate the experimental data more effectively, the average axial bolt strains were normalized by the maximum applied load for each test, since the strains perform linearly with respect to the applied load. Then these unit strains were normalized by the predicted individual bolt strain for a unit applied load. Table 3.3 demonstrates this procedure for the plotted data shown in Figure 3.3. If the predicted anchor bolt stresses were calculated using an accurate model, the normalized strains at all bolt locations would be expected to equal 1.0. A statistical study was then conducted to evaluate these normalized average axial bolt strains at the maximum applied tension and compression loads. Due to the averaging of the strain gage readings per bolt, the predicted strains in bolts #3 and #7 are zero. Because the measured strains are also near zero in bolts #3 and #7 and the normalization process results in numbers close to infinity or zero, the data from these bolts are excluded from the statistical analysis.

At first, it was not clear if the results using the data from bolts #2, #4, #6, and #8 (closer to the neutral axis) would be different from the results from bolts #1 and #5 where the average axial strains were higher. For tests conducted with a 1.5 in base plate with no applied torsion, the mean and variance for the data from each set of bolts were compared to see if any significant differences existed. From basic statistics, the mean is defined as the arithmetic average of the data set while the variance measures how closely the data are clustered about the mean value. For bolts #1 and #5 only, the mean and variance were 1.09 and 0.029 respectively. When these data are included in the data set with bolts #2, #4, #6, and #8, the mean and variance of the data were 1.054 and 0.024 respectively. Even though the mean for bolts #1 and #5 is slightly larger than for the set with all six bolts, the variances of the two sets are fairly similar. Therefore, from this information, it can be assumed that, by including all six bolts into one large data set, no significant differences occur. For further proof of this assumption, a "t-test" was conducted on the mean and an "F-test" was conducted on the variance. These tests concluded, with greater than a 95 percent confidence level, that the means and variances from these two data sets were not significantly different. (These tests are explained in standard statistical references [5].)

In the tests conducted with misalignment and torsion, a similar close agreement was found between the data set for bolts #1 and #5 and the data set for all six bolts. Therefore, it was concluded that the data from all six of these bolts could be considered as one consistent data set or "population". Table 3.4 is a summary of statistical data (i.e. mean and variance) for all six bolts and for bolts #1 and #5 alone for comparison. The effects of torsion on the distribution of bolt stresses can be seen by comparing the results at M:T:V ratios of 19.5:5:1 and 14.5:10:1 to those at M:T:V ratios of 19.5:0:1 and 14.5:0:1. There is only a random difference in the means of these data sets, meaning the torsion did not increase the magnitude of the average axial stresses in the bolts systematically. However, the averaged data sets including torsion had variances approximately three times greater than those without torsion. F-tests show that there is greater than 99 percent confidence that the variance of the data with torsion is greater than the variance of the data without torsion. Due to inconclusive results, the tests conducted with the 1 in base plate on a misaligned anchor bolt pattern do not show this increase in variance due to torsion.

The increase in variance due to torsion can also be seen in Figure 3.4 which presents histograms of the normalized bolt strains for tests with a 1.5 in base plate and no misalignment, with and without torsion. For tests with torsion, the normalized bolt strain data were distributed from a low of 0.4 to a high of 1.6 with approximately 25 percent of strains clustered near 90 percent of the theoretical bolt strain. Normalized bolt strain data for tests without torsion were distributed from a low of 0.6 to a high of 1.3 with approximately 42 percent of the results clustered near 90 percent of the theoretical bolt strain.

The base plate thickness also had a significant effect on the anchor bolt stresses. The data in Table 3.4 seem to suggest that the mean has increased for the case of the 1 in thick base plate without misalignment, although this could not be established with confidence by t-tests. However, the F-tests showed that the 1 in thick base plate increased the variance, with greater than 97 percent confidence in all cases except the case of misalignment and torsion. For the misaligned test for this case, the variance did not increase with torsion and seems anomalous.

It is believed that an increase in flexibility of the base plate causes localized bending to occur in the anchor bolts. This localized bending, or prying, is caused by a non-uniform distribution of stresses on the anchor bolt nut which would increase scatter in the test data. Based on these findings, only adequately stiffened base plates should be allowed in the design of cantilevered support structures. It appears that selecting a base plate thickness equal to, or greater than, the anchor bolt diameter is a reasonable rule of thumb.

Surprisingly, misalignment did not have a statistically significant effect on the results. Based on Table 3.4, it appears that the mean is lower for the misaligned data, although no rational explanation for this bias has been developed. Based on the data for the 1.5 in thick base plate only, it also appears that misalignment increases the variance and intensifies the increase in variance due to torsion. However, none of these apparent differences could be established with confidence using the statistical tests. Therefore, it was concluded that the misalignment of the anchor bolt pattern (1:20) does not significantly influence the distribution of stresses among the bolt group. Misalignments of less than 1:20, which are more realistic installation conditions, will most likely have an even smaller effect on the distribution of bolt stresses.

At least one test on each assembly was conducted after loosening bolt #1 from the anchor bolt pattern so that no load would be sustained by this bolt. The objective of these tests was to simulate the effect of a completely loose or fractured bolt. Based on the simple flexure model, it would be expected that strains in anchor bolts #2 and #8 (i.e. those adjacent to anchor bolt #1) would increase by 69 percent as the load is transferred to these two bolts. The test results however, indicated that the average increase in strain at these two locations was 51 percent. Therefore, M*c/I design assumption conservatively overestimates the distribution of strains for this condition. It is believed that with one or more bolts missing from the bolt pattern, flexibility of the base plate affects the distribution of forces to the anchor bolts. A single test was also conducted to simulate the behavior of a four bolt anchor bolt pattern. In addition to the two bolts located on the neutral axis of bending (bolts #3 and #7), bolts #1 and #5 were also completely loosened to produce a four bolt pattern. Unfortunately, the results of this test were inconclusive with respect to determining the distribution of anchor-bolt stresses. Excessive distortion in the 1.5 in base plate caused the plate to bind up on these loosened bolts and effectively transfer axial loads. These observations highlight the importance of a properly sized and detailed base plate in effectively distributing support-structure forces to the anchor bolt assembly.

Therefore, in order to examine the probable distribution of anchor bolt stresses, only data from the 1.5 in thick base plate needs to be considered. A histogram of normalized bolt strains for all tests (with and without misaligned anchor bolts) conducted on specimens with a 1.5 in base plate is presented in Figure 3.5. As shown in the figure, the data appear to be normally distributed. The mean and variance of this data set are 0.93 and 0.064 respectively. Since there is no systematic effect on the mean which is approximately 1.0, it appears reasonable to calculate the relationship between anchor-bolt stresses and support-structure forces with the simple flexure formula (M*c/I). The variance of the data will contribute to the overall uncertainty in fatigue resistance. However, for balance, for every bolt with a stress range on the high side there should be a bolt with a stress range on the low side. This level of variance in the measured vs. theoretical stresses is not any greater than the variance in measured vs. theoretical stresses for other structural details. Also, this uncertainty in fatigue resistance is far less than the uncertainty in the magnitude of the wind loads discussed in Section 2.2 [2]. Therefore, it is concluded that no additional factors must be included in the design procedure to account for the variance in the bolt stresses. Furthermore, the variance in the bolt stresses will have no effect on the ultimate strength. Although certain bolts may begin to yield prematurely, the ductility will allow full strength to be obtained from each bolt.

Bending - In addition to the axial stresses generated from resisting overturning moments at the cantilevered support structure base, anchor bolts will also be subject to bending stresses. Bending moments in individual anchor bolts are typically generated by the transfer of horizontal base shear forces and torsional moments as individual shear forces applied through the base plate to the anchor bolts and into the concrete foundation. As expected, the bending stresses are most significantly affected by the amount of exposed bolt between the top of the foundation and bottom of the leveling nut, assuming that the bending effects between the nuts are minimized. Standard plans for cantilevered support structures typically specify that exposed lengths shall not exceed 1 in, which minimizes the moment arm for bending from these shear forces. However, some states tend to dry-pack grout under the base plate, which requires a greater exposed length. In addition to increasing the bending stresses, the dry pack has been reported to be a source of corrosion problems as it cracks and retains water. For these reasons, grouting between the base plate and the concrete should be discouraged.

Assuming elastic behavior and plane sections, the amount of bending in individual anchor bolts was evaluated by closely examining the data for each of the four strain gages placed on a typical anchor bolt. By comparing measurements at two strain gage locations 180 degrees apart on an anchor bolt, the calculation of bending moments about a perpendicular axis was possible. The biaxial moments were then resolved into the principal moments using flexure theory.

From flexure theory the curvature (Φ) and axial deformation described by the strain plane can be calculated, where $M = \Phi^* EI$. For example, measurements obtained from a test with a 1.5 in base plate, straight anchor bolt pattern, and a 3 in exposed length resulted in the computation of bending moments of 1.09 and 0.78 kip-in at anchor bolts #3 and #7 respectively. (These bolts near the neutral axis of the bolt group had very little axial force and were therefore most suitable for measuring bending stress).

Since a significant factor in determining the bending stresses is the exposed bolt length, it is expected that the placement of the gages on the anchor bolt shaft would affect the stresses. However, because the strain gages on the bolt shank were placed so close to the top of the concrete foundation, the length of the beam in the model is much larger than the distance between the center of the gage and the top of the concrete foundation. Therefore, the bending stresses measured at the gage can be assumed to be equivalent to the maximum bending stresses. Although, it is possible that this distance is a large percentage of the exposed length for shorter exposed lengths and will be taken into consideration during analysis.

The best model to use to calculate the bending stresses was found to be a fixed-fixed beam, with one end at the top of the concrete and the load acting at the midpoint of the beam (the middle of the base plate). Referring to Figure 2.4, the beam length would be either 5.25 in or 3.25 in for an exposed length of 3 in or 1 in, respectively. It is assumed that the top end is free to displace circumferentially around the bolt circle relative to the end in the concrete. Assuming a horizontal shear of 1/8 the applied load and a beam length of 5.25 in, a theoretical bending moment was calculated which is 40 to 90 percent greater than the measured moment. It is recognized that the assumption of a fixed connection at the concrete interface is not consistent with the actual behavior at this location. Some amount of elastic and inelastic deformations will occur along the anchor bolt-to-concrete interface which may cause cracking or spalling at high shear loads. In fact, tests on the misaligned anchor bolt assembly resulted in some limited spalling in this area. However, the use of a greater length (to account for fixity at some point below the top of concrete or to take the point of load application as the mid-thickness plane of the base plate) makes the theoretical moment even greater which makes the result less accurate. Similarly, the use of one or two pinned ends for the beam model would provide an even more conservative result and was therefore not considered.

Correlation of measured and predicted anchor bolt moments for a similar test configuration with only a 1 in exposed length proved to be even more inaccurate. It is believed that the short length of exposed bolt (less than one diameter) was inadequate to develop bending stresses (the bolt behaved more like a shear link). Although a comparison could not be made to theoretical bending stresses, the magnitude of these stresses could be evaluated from the test data. For the most critical bolts in axial tension (bolts #1 and #5) the percentage of bending stress was less than 10 percent of the average axial stress. This increase in stress is small and occurs at only one location on the bolt circumference. There is a lower combined stress range at the point 180 degrees away. Therefore, it appears that the bending effect can be ignored for anchor bolts which have an exposed length less than one diameter in height. Also, as long as the strain gages are placed on the anchor bolt shank within a width of the gage to the concrete, the stresses measured from the gages can be assumed to be the maximum stress and no gradient must be considered. Exposed heights greater than one bolt diameter, for grouting for example, would require calculation of the bending stress using the conservative fixed-fixed beam model.

3.5.3.2 Column Base

By analyzing the strain data from the gages on the column stub, the overturning moment at the base can be verified. Although the majority of the strains measured on the column were consistent with analytical predictions, the calculation of longitudinal bending stresses in the column, 2 in above the base plate, required the use of biaxial stress/strain relationships. Significant transverse (i.e. hoop) forces generated by the restraint provided by the socket connection weld caused a transverse strain component to be generated.

Tests conducted on Assembly #1 (1.5 in thick base plate and no misalignment) only measured the uniaxial strains on the column at 4 in above the base plate. Figure 3.6 shows the distribution of uniaxial strains in the column for both the theoretical and experimental strains using uniaxial calculations and strain gages. The strains measured at intermediate points along the length of the column between the applied load and the base plate were consistent with the predicted strains. However, the strain gages about 19 ft below the applied load, or 4 in above the base plate, recorded strains one half the predicted values. From these results, it was determined that biaxial gages and stress/strain equations were necessary to determine the stresses in the column near the welded joint.

As shown in Table 3.5, the predicted stresses for an applied unit load are compared to normalized maximum X-X axis stresses at 2 in and 4 in above the base plate on Assembly #2, #3, and #4 for all four M:T:V ratios. Assembly #1 results are not included in this table because only uniaxial strains were recorded. Also, for Assembly #3, a biaxial rosette was not placed at 4" above the base plate. Therefore, the relationship at this height cannot be determined. By examining Figures 3.7, 3.8, and 3.9, a qualitative relationship of the results from the tests represented by the data in the table can be determined.

All three figures depict the results for the M:T:V ratio of 19.5:0:1, where no torsion was applied. Figure 3.8 shows the good correlation between the theoretical and the measured values of stress for the 1.5 in plate, while the 1 in plate increases the stress closer to the welded joint in relation to the theoretical stress as shown in Figure 3.9. Looking at the maximum values on Figure 3.7 and 3.9, misalignment seems to slightly decrease the stresses in the column.

The effect of base plate thickness, misalignment and torsion can be determined quantitatively from the data given in Table 3.5. Since the strain gages were placed very close together, the difference between the predicted stresses at 2 in and 4 in above the base plate is minimal. However, for the measured stresses, the difference is more significant, especially for Assembly #2 and #4. First considering tests conducted without torsion, the actual stress at 2 in is about twice the predicted, while the stress at 4 in is less than half that predicted. This significant difference can be attributed to the undersized base plate of 1 in, since the actual stress at 2 in for Assembly #3 is very consistent with the predicted stress. These conclusions are consistent with those found from the anchor bolt data in that the 1 in base plate is inadequate for cantilevered support structures. It is possible that the deformation of the 1 in base plate introduces secondary bending in the column creating a reversal of stresses between 2 in and 4 in above the base plate. A schematic of this probable distortion and the resulting reversal in deflection of the column is shown in Figure 3.10.

Torsion and misalignment appear to have minimal effect on the stress in the column base. For the tests conducted on Assembly #2 (1 in base plate and no misalignment), torsion does increase the stresses as expected. Misalignment, on the other hand, seems to decrease the stress in the column, depending on which situation is analyzed. Even though the predicted results with misalignment are minimally different compared to those without misalignment, this reaction is the opposite of the expected result of a slight increase in stress. However, for the test conducted on Assembly #4 (1 in base plate with misalignment), the effect of torsion seems to be influenced by the misalignment.

3.6 Fatigue Tests

3.6.1 Loading Procedure

At the conclusion of the 32 static tests on the column-anchor bolt assemblies, four full-scale proof-of-principle fatigue tests were conducted on the two bolt assemblies with a 1.5 in thick base plate used in the full-scale concrete foundations to verify results of the anchor bolt fatigue tests described in previous research [2]. With the actuator positioned such that only horizontal shear and overturning moment were applied to the eight bolt group (i.e. no torsion), a cyclic load was applied to the specimen to simulate loads produced by wind-induced vibrations.

Two tests were conducted on the concrete foundation cast with misaligned anchor bolts (1:20), and two tests were conducted on the foundation cast without misalignment. For all tests, the exposed anchor bolt length was set at 1 in to reflect typical installation conditions. However, the bolts were only snug-tightened to allow comparison with the anchor bolt fatigue tests data described above. The actuator load was applied at a frequency of 1 Hz in only one direction such that a 20 ksi tension-tension stress range was measured in the critical bolt (bolt #1 or #5). The minimum bolt stress was set to accommodate the effect of the dead load, while the maximum bolt stress was set at approximately 60 percent of the minimum yield stress (0.6^*F_y) to duplicate the conditions used in the individual anchor bolt fatigue tests. After failure of the first bolt on each concrete foundation, the direction of loading was reversed and the other critical bolt was similarly tested.

3.6.2 Anchor Bolt Fatigue Prediction

Although some tests have been done through other research, they have concentrated on subjecting anchor bolts to relatively high stress ranges only. Frank's [6] studies suggest that anchor bolts preloaded to one-third turn past sung and loaded in direct tension or non-preloaded and loaded in torsional bending exhibit fatigue strengths comparable to AASHTO Category C while non-preloaded anchor bolts loaded in axial tension exhibit fatigue strengths comparable to AASHTO Category E.

Through previous research associated with this project, fatigue tests were conducted on individual anchor bolts in order to determine the CAFL for anchor bolts in axial tension. It was concluded that the AASHTO Category E' design curve should be used to design axially-loaded, sung-tight anchor bolts in the regime of finite life. However, for the design of axially-loaded anchor bolts tightened to one-third-of-a-turn beyond sung, the AASHTO Category E design curve should be used. Also, when considering the regime of infinite life, the CAFL corresponding to AASHTO Category D (7 ksi) should be used to design axially-loaded, snug- and fully-tightened anchor bolts. To determine if the individual tests were a good representation of full-scale conditions, full-scale fatigue tests were conducted under similar stress ranges in the infinite life regime.

Another aspect of failure caused by fatigue that needs to be considered is the propagation of the crack. According to Van Dien's research [2], failure of the individual bolts was caused by the propagation of a single crack which formed at the first fully-engaged thread from the loaded face of the exterior nut of the double-nutted connection. Inspection of the failure surfaces indicated that fatigue cracks initiated at multiple points at the thread root that coalesced into a single primary crack which resulted in failure. This prolonged "initiation" or coalescence period is the primary source of the effect of yield strength on anchor bolt fatigue strength. In all situations, the primary crack propagated through approximately 65% to 75% of the specimen cross-section prior to fracture. The placement of the crack on the bolt and the amount of propagation prior to fracture will also be verified through the full-scale testing.

3.6.3 Results

3.6.3.1 Anchor Bolts

Fatigue failures in the straight anchor bolts occurred at 263,000 and 397,000 cycles while failures in the misaligned anchor bolts occurred at 240,000 and 488,000 cycles. A plot of the results from these four full-scale foundation tests, subjected to a tensile stress range of 20 ksi, is presented in Figure 3.11 and compared to the snug-tight data obtained from the individual anchor bolt fatigue tests conducted by Van Dien [2]. In accordance with the recommendation determined through the static tests, bending stresses were not considered in plotting the stress ranges for these four data points in Figure 3.10. The four data points from the foundation tests conducted on bolts with and without misalignment are within the scatter band of results for individual bolts tested without misalignment. Although the data set is not large, it is reasonable to conclude that the results obtained from individual anchor bolt fatigue tests accurately predict the behavior of bolts in a complete assembly. Furthermore, the fact that the four points are within the scatterband also suggests that it is not necessary to include the bending stresses in the stress range for exposed lengths less than one bolt diameter.

In addition, these full-scale foundation tests indicated that misalignment did not significantly affect the fatigue resistance. Although the amount of misalignment was well beyond typical installed conditions (i.e. 1:20), the two misaligned data points were within the level of scatter. One misaligned bolt actually exhibited the longest fatigue life. It is possible that the compression cycles that this bolt experienced during the first test in the other direction could limit the fatigue life in the second test, but no such bias was observed in the second test results relative to the first test results. Since these tests were conducted at high stress ranges, it is also very possible that the prying effects due to the misalignment were not as severe as is possible in low stress ranges. Until tests are conducted at lower stress ranges, there is still uncertainty regarding the effect of misalignment on the fatigue strength of anchor bolts.

The propagation of the fatigue cracks in the anchor bolts were consistent with that from the individual fatigue tests. A single crack formed at the first fully-engaged thread from the loaded face of the exterior nut of the double-nutted connection and coalesced about 70% across the cross-sectional area prior to complete failure. The bolt also exhibited its ability to provide resistance even after the crack formed. Figure 3.12 is a closeup photograph of an extreme bolt after fatigue, showing the location of where the fatigue crack occurred in relation to the base plate as well as the failure surface of the bolt.

3.6.3.2 Column Base

As a result of these full-scale anchor bolt assembly fatigue tests, information was also gained on the fatigue resistance of two common column-to-base plate connection details. As mentioned above, the fatigue test was initiated with a double-fillet welded socket connection. The applied loads necessary to propagate fatigue cracks in the anchor bolts generated stress ranges of 15 ksi at the column base which led to the development of a fatigue crack at only 24,000 cycles. Note that this data point falls below the Category E' S-N curve which is estimated for socket connection details.

To complete the fatigue test program, the socket connection was retrofitted with the addition of eight gusset stiffeners designed as a Category C detail. The stiffeners were fabricated from 0.5 in thick plate and extended 24 in up the column shaft with the outstanding edge of the stiffener plate ground and taper transitioned at less than a 15 degree angle from the wall of the column. A full-penetration weld with fillet reinforcement was used for 6 in near the termination of the stiffener. Now the nominal stress range at the column base is still 15 ksi with a stress range of about 13 ksi at the gusset. With these modifications, approximately 1.4 million cycles were accumulated on the specimen without any fatigue cracks. The fatigue strength of this improved detail can be estimated by the Category C S-N curve.

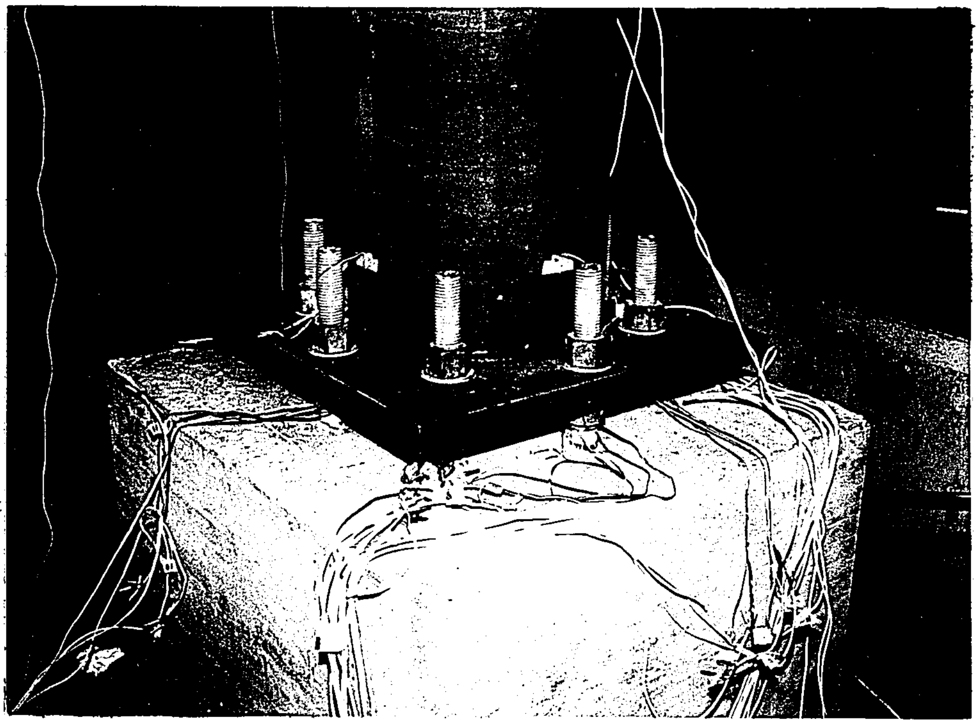


Figure 3.1 Photograph of Column Base/Anchor Bolt Assembly

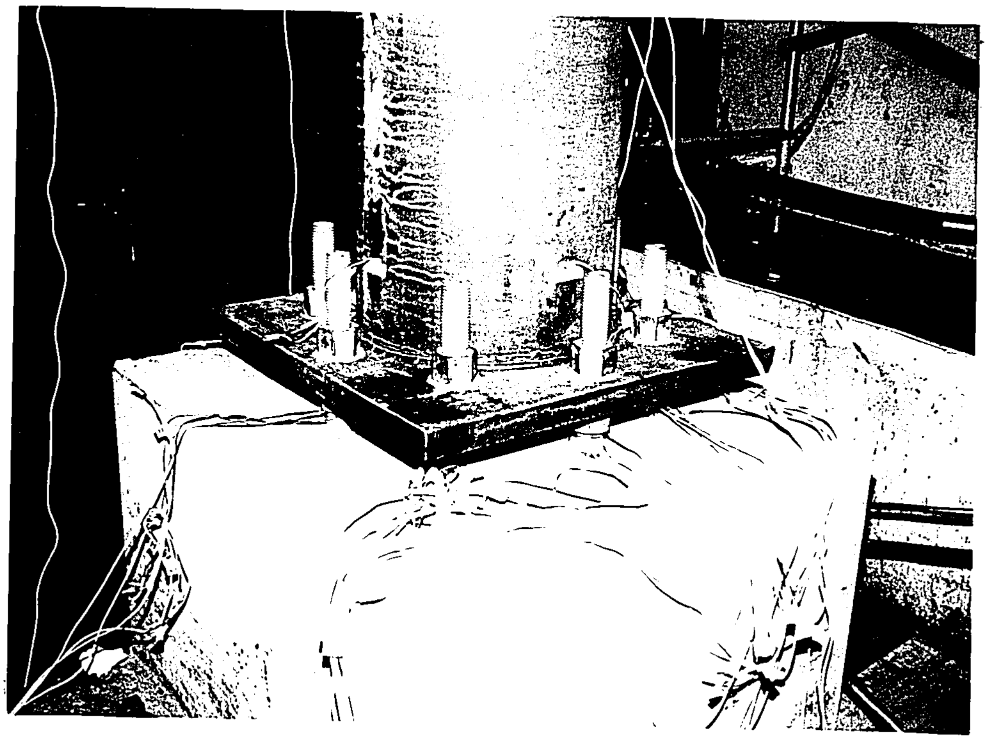
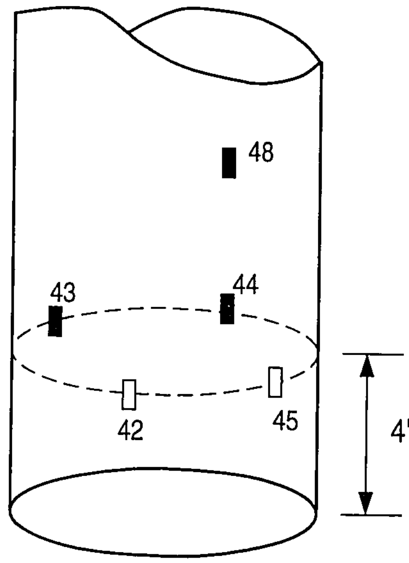


Figure 3.1 Photograph of Column Base/Anchor Bolt Assembly

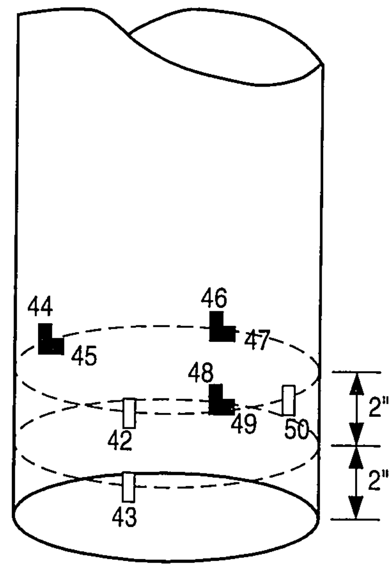
Column Instrumentation Placement

Assembly #1



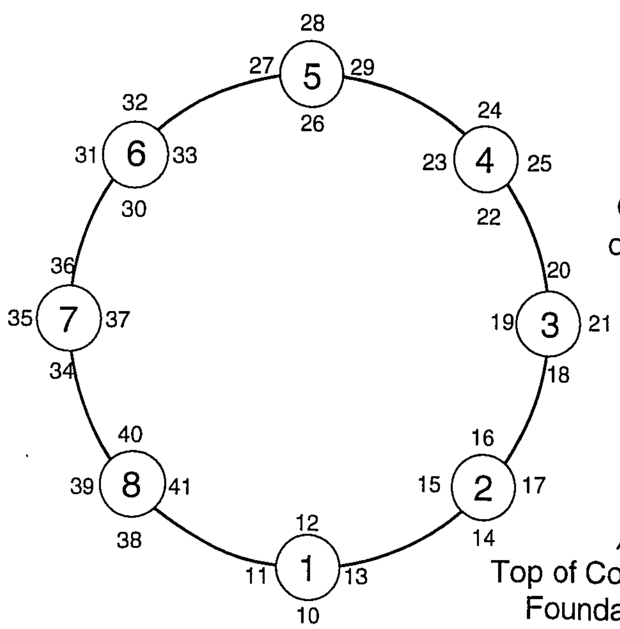
48 @ 2' above base plate
49 @ 4' " " "
50 @ 12' " " "

Assembly #2, #3 & #4



Note: 42 is above bolt 1

Anchor Bolt Instrumentation Placement



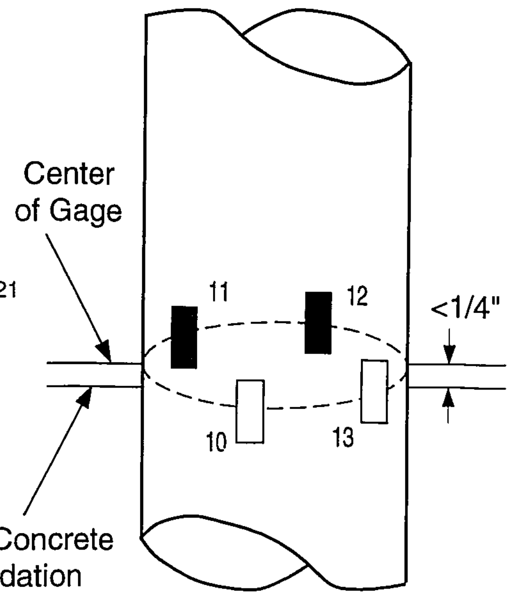


Figure 3.2 Instrumentation Placement

Predicted Stresses (ksi) and Corresponding Strains (Microstrain)				
Bolt #	P1	P2	P3	P4
1	3.671 126.6	2.741 94.6	3.71 127.9	2.92 100.7
2	2.591 89.3	1.938 66.8	2.65 91.4	2.18 75.2
3	0.0813 2.80	0.0813 2.80	0.545 18.8	1.01 34.8
4	2.591 89.3	1.38 66.8	2.65 91.4	2.18 75.2
5	3.671 126.6	2.741 94.6	3.71 127.9	2.92 100.7
6	2.591 89.3	1.938 66.8	2.65 91.4	2.18 75.2
7	0.0813 2.80	0.0813 2.80	-0.383 -13.2	-0.848 -29.2
8	2.591 89.3	1.938 66.8	2.65 91.4	2.18 75.2

Predictions done for Assembly #1: All bolts tight, 1.5 in base plate, no misalignment, 1 in exposed length

Positions #1-4: Represent the four M:T:V ratios (19.5:0:1, 19.5:5:1, 14.5:0:1, 14.5:10:1, respectively)

Position #1 - M:T:V 19.5:0:1
Position #2 - M:T:V 19.5:5:1
Position #3 - M:T:V 14.5:0:1
Position #4 - M:T:V 14.5:10:1

Table 3.1 Predicted Bolt Stresses and Strains for Entire Bolt Group with a Unit Load applied at Position #1-4 on Assembly #1

	M:T:V	Cantilever Model	Fixed-Fixed Model
1"	19.5:0:1 or 14.5:0:1	V = 0.125	V = 0.125
		M = 0.406	M = 0.203
3"	19.5:0:1 or 14.5:0:1	V = 01.25	V = 0.125
		M = 0.656	M = 0.328
1"	19.5:5:1	V = 0.1845	V = 0.1845
		M = 0.600	M = 0.300
3"	19.5:5:1	V = 0.1845	V = 0.1845
		M = 0.969	M = 0.484
1"	14.5:10:1	V = 0.244	V = 0.244
		M = 0.793	M = 0.396
3"	14.5:10:1	V = 0.244	V = 0.244
		M = 1.281	M = 0.641

Note: All calculations were performed for a Unit Applied Load

All Forces are in kips

All Moments are in kip*in

Cantilever Model: $M = V * L$

Fixed-Fixed Model: $M = (2 * V) * (2 * L) / 8$

Table 3.2 Predicted Bending Moments in Bolts Induced by Shearing Forces

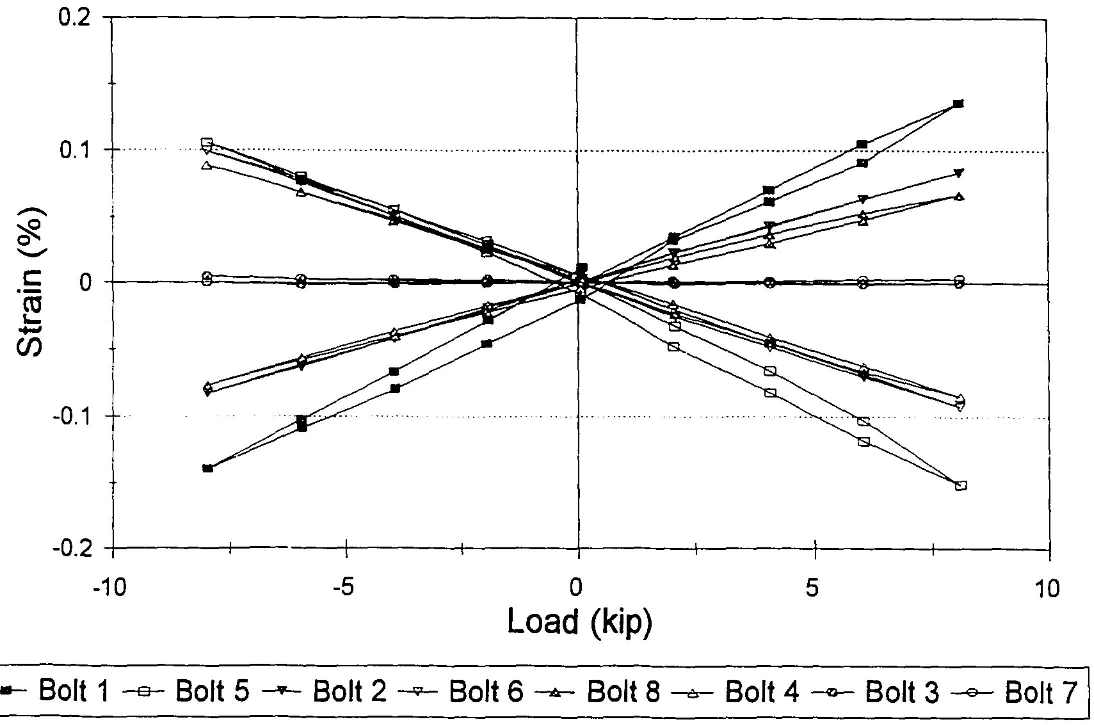


Figure 3.3 Distribution of Axial Strains in Bolt Group with 1.5 in Base Plate (Tests without Misalignment or Torsion and 1 in Exposed Length)

Bolt #	1	2	3	4	5	6	7	8
Distance from Neutral Axis (in)	10.5	7.42	0	7.42	10.5	7.42	0	7.42
Average Measured Strain (Microstrain)	1382	838	25	863	1280	958	19	720
Normalized to Maximum Load	173	105	3.2	107	160	119	2.5	90
Normalized to Predicted Strain for each bolt	1.35	1.16	NA	1.18	1.21	1.31	NA	0.99

Results of Normalization of Strains from the test conducted on Assembly #1 with an M:T:V of 19.5:0:1 and an exposed bolt length of 1 in

The maximum load for this test was 8 kip.

Table 3.3 Normalization Process of Measured Strains to Predicted Strains

		Bolts #1,2,4,5,6&8		#1 & #5 only		
Plate Size (in)	M:T:V Ratio	Mean	Variance	Mean	Variance	Misaligned Bolt Pattern
1.5	19.5:0:1 and 14.5:0:1	1.05	0.024	1.09	0.029	N
1.5	19.5:5:1 and 14.5:10:1	1.01	0.064	1.11	0.031	N
1.5	19.5:0:1 and 14.5:0:1	0.83	0.033	0.93	0.001	Y
1.5	19.5:5:1 and 14.5:10:1	0.83	0.102	1.05	0.081	Y
1	19.5:0:1 and 14.5:0:1	1.23	0.055	1.30	0.051	N
1	19.5:5:1 and 14.5:10:1	1.06	0.174	1.31	0.169	N
1	19.5:0:1 and 14.5:0:1	0.85	0.058	0.76	0.047	Y
1	19.5:5:1 and 14.5:10:1	0.76	0.055	0.76	0.109	Y

Table 3.4 Summary of Statistical Data:
All Six Bolts versus Bolts #1 & #5 Only

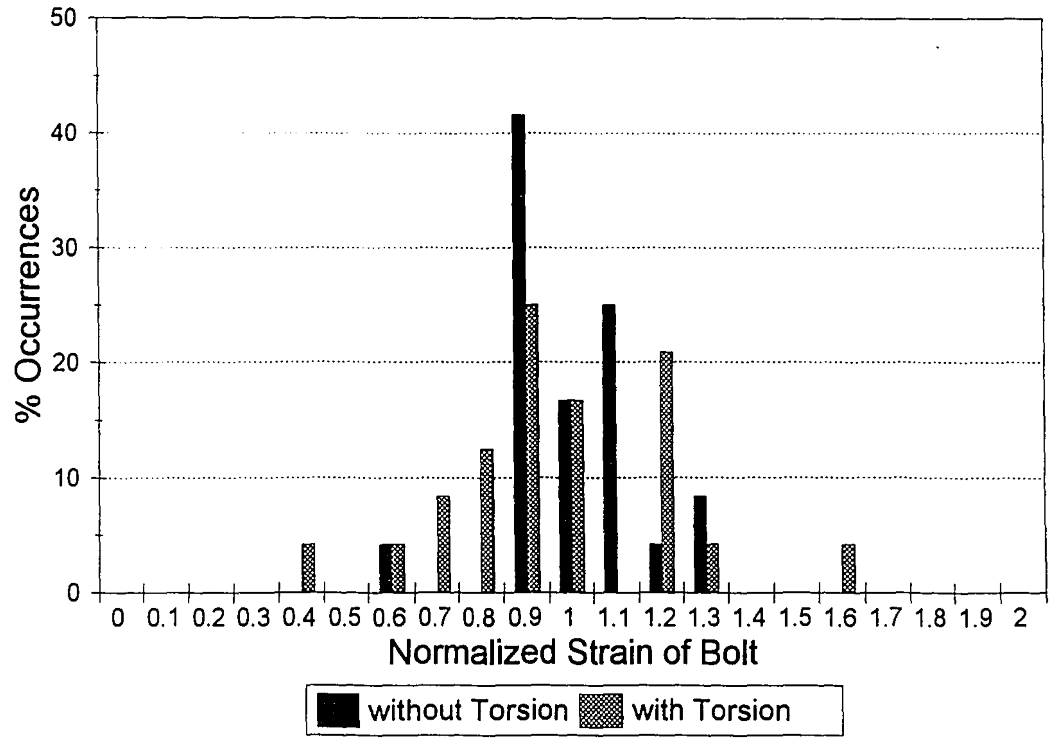


Figure 3.4 Histogram of Normalized Bolt Strains (No Misalignment)

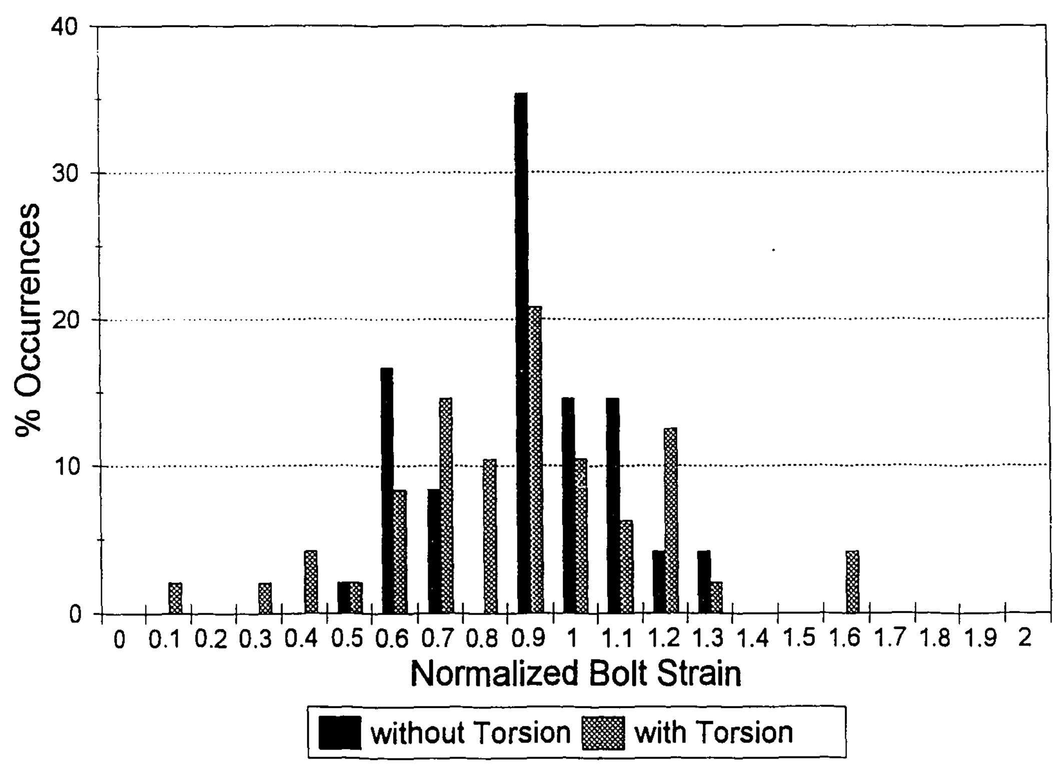


Figure 3.5 Histogram of Normalized Bolt Strains
(All Data with 1.5 in Base Plate)

			2" Above Base Plate		4" Above Base Plate	
M:T:V	Base Plate Size (in)	Misaligned Bolt Pattern	Predicted Stress (ksi)	Actual Stress (ksi)	Predicted Stress (ksi)	Actual Stress (ksi)
Assembly #2						
19.5:0:1	1	N	2.55	4.25	2.52	0.727
14.5:0:1	1	N	1.89	3.5	1.86	0.638
19.5:5:1	1	N	2.58	5	2.55	0.9012
14.5:10:1	1	N	1.894	3.67	1.865	0.676
Assembly #3						
19.5:0:1	1.5	Y	2.55	2.39	NA	NA
14.5:0:1	1.5	Y	1.88	1.75	NA	NA
19.5:5:1	1.5	Y	2.55	2.99	NA	NA
14.5:10:1	1.5	Y	1.89	2.29	NA	NA
Assembly #4						
19.5:0:1	1	Y	2.56	4.37	2.53	0.938
14.5:0:1	1	Y	1.89	3.31	1.86	0.569
19.5:5:1	1	Y	2.55	4.14	2.52	0.555
14.5:10:1	1	Y	1.894	3.66	1.865	0.466

Note: Exposed bolt length does not affect the stresses in the column

Predicted Longitudinal Stresses calculated using Moment, Torsion, and Shear components, including bending

Equations used to Calculate the Longitudinal Stress

$$\sigma_x = 1.1 * E (\varepsilon_x + 0.3 * \varepsilon_y)$$

Table 3.5 X-X Axis Stresses in Column Base with a Unit Applied Load for Assemblies #2, #3, and #4

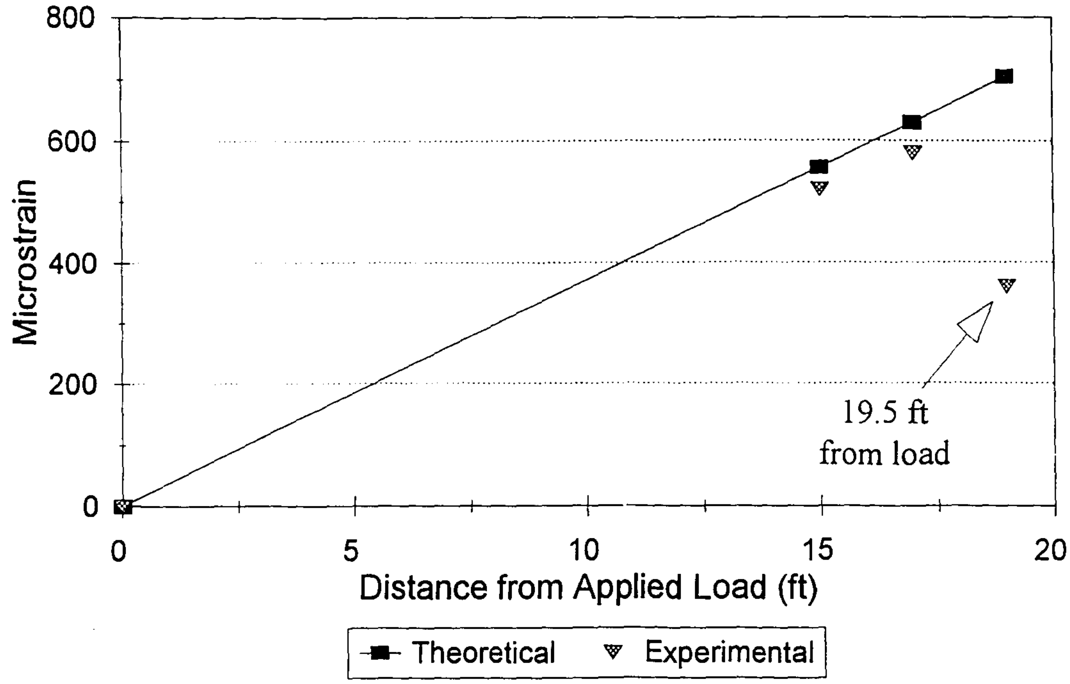


Figure 3.6 Distribution of Uniaxial Strains in Column for Assembly #1

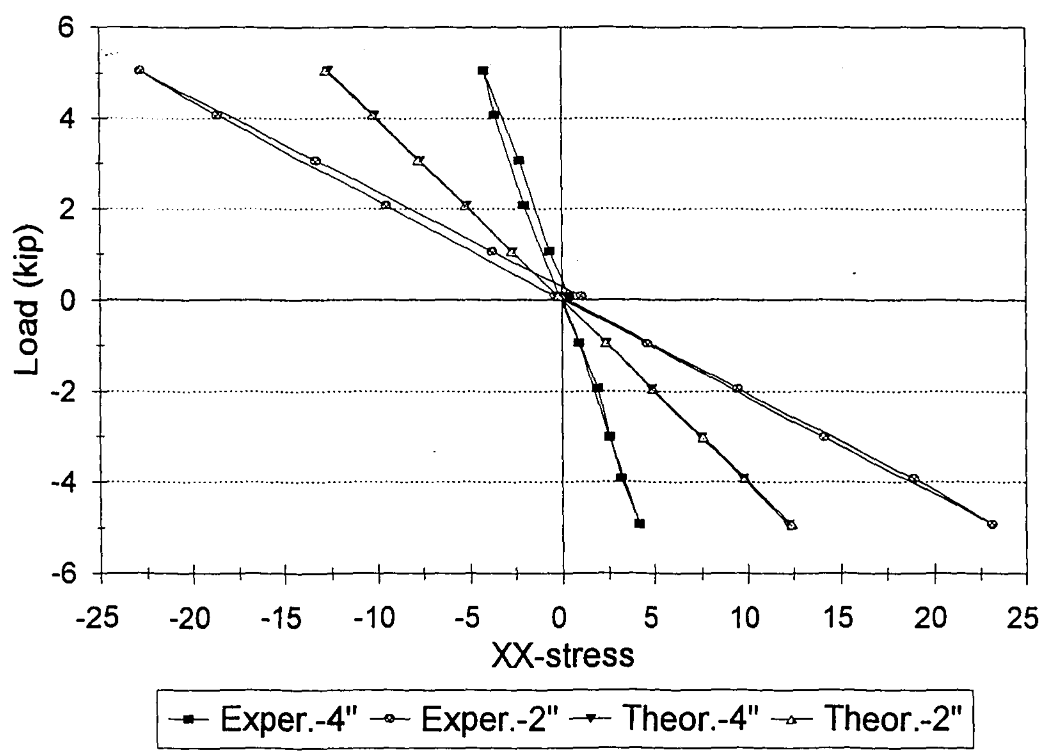


Figure 3.7 Distribution of Biaxial Stresses in Column for Assembly #2

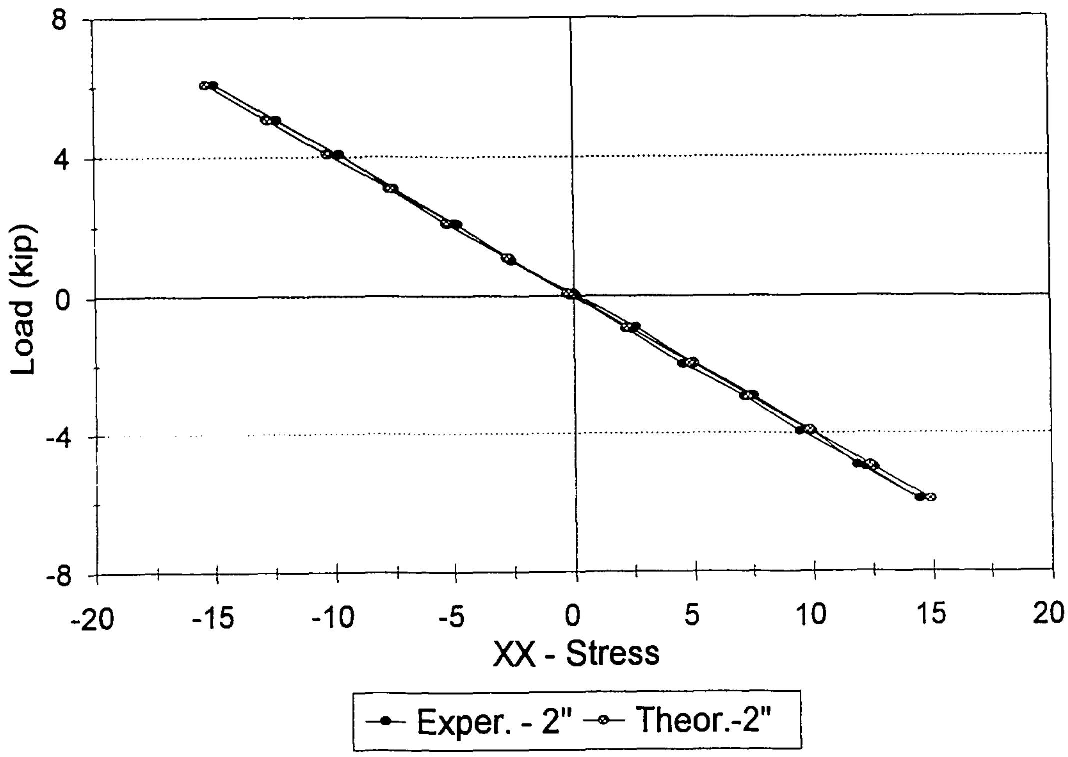


Figure 3.8 Distribution of Biaxial Stresses in Column for Assembly #3

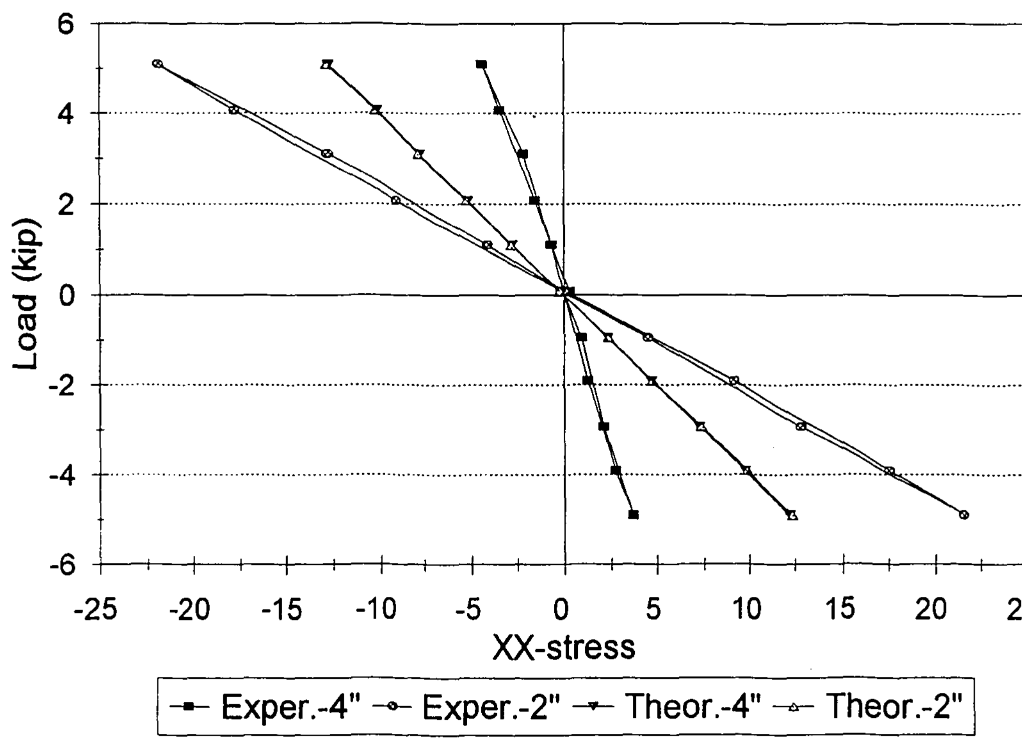


Figure 3.9 Distribution of Biaxial Stresses in Column for Assembly #4

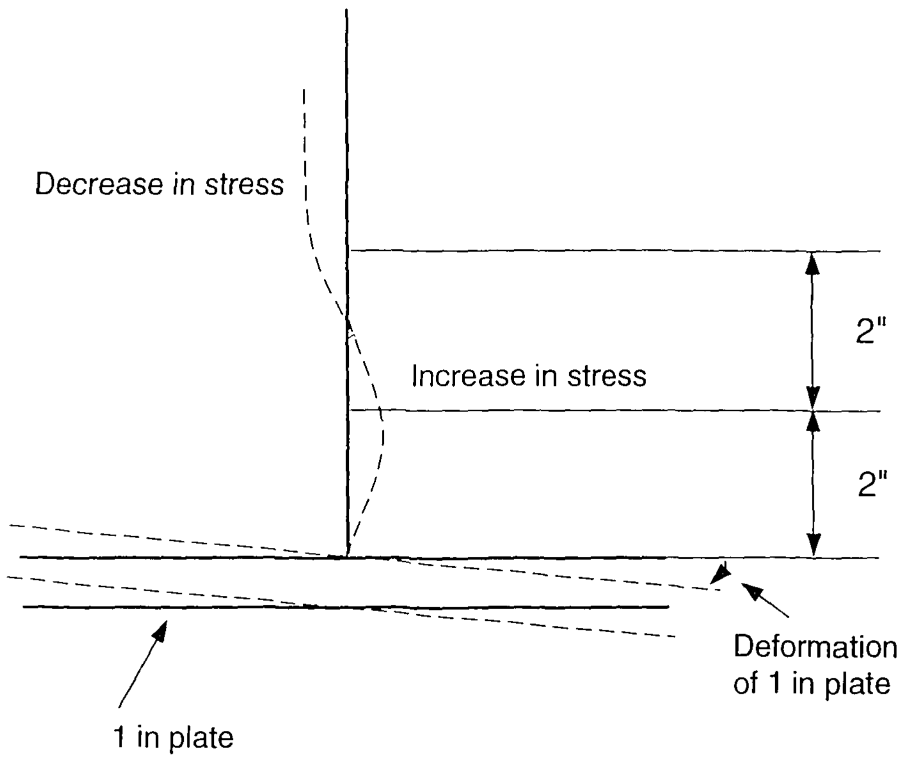


Figure 3.10 Possible Reversal of Stresses in Column
due to Deformation of 1 in Base Plate

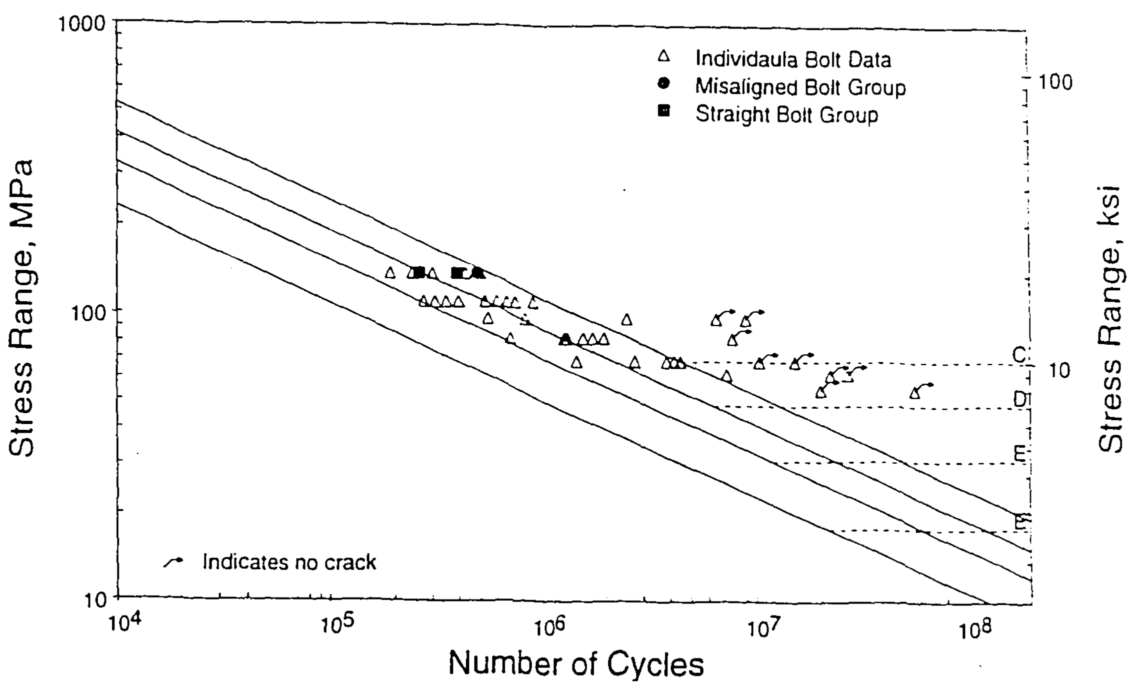


Figure 3.11 S-N Curve of Anchor Bolt Assembly Test Data
and Individual Bolt Test Data

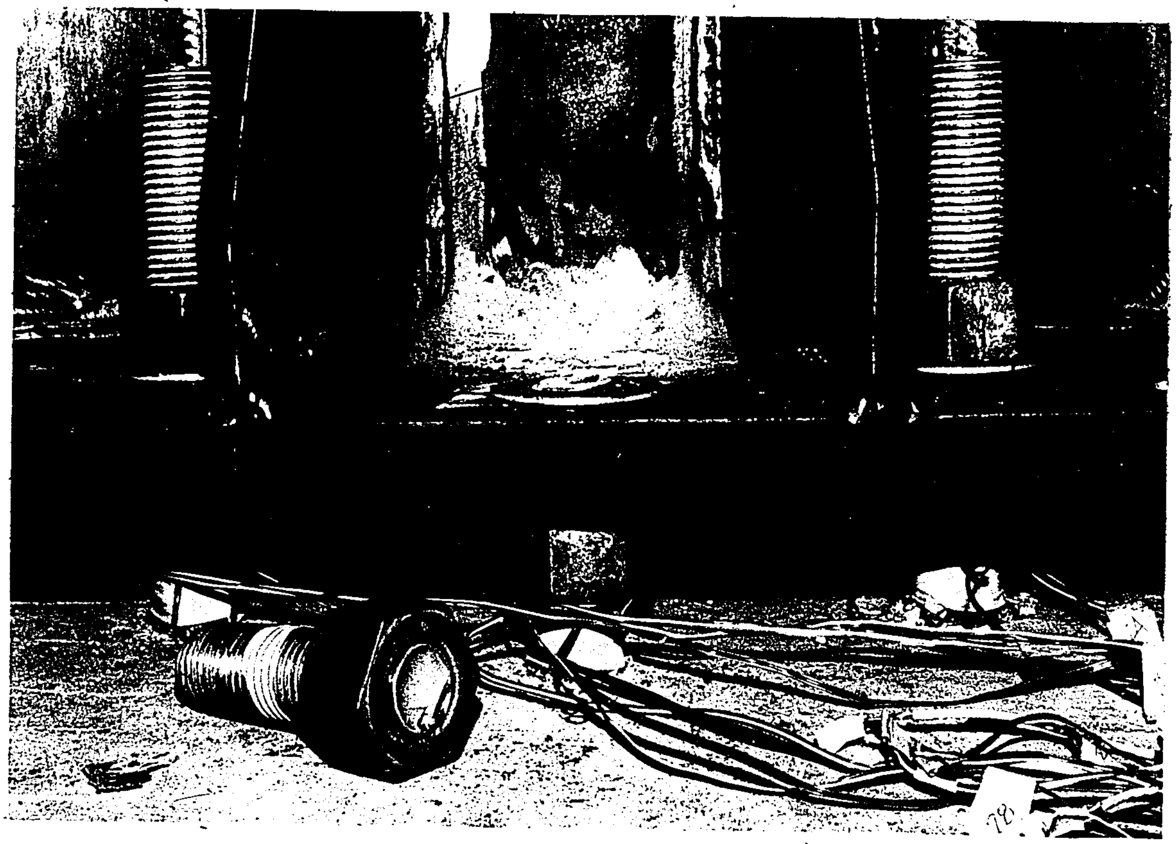


Figure 3.12 Closeup Photograph of Failure Location at Anchor Bolt

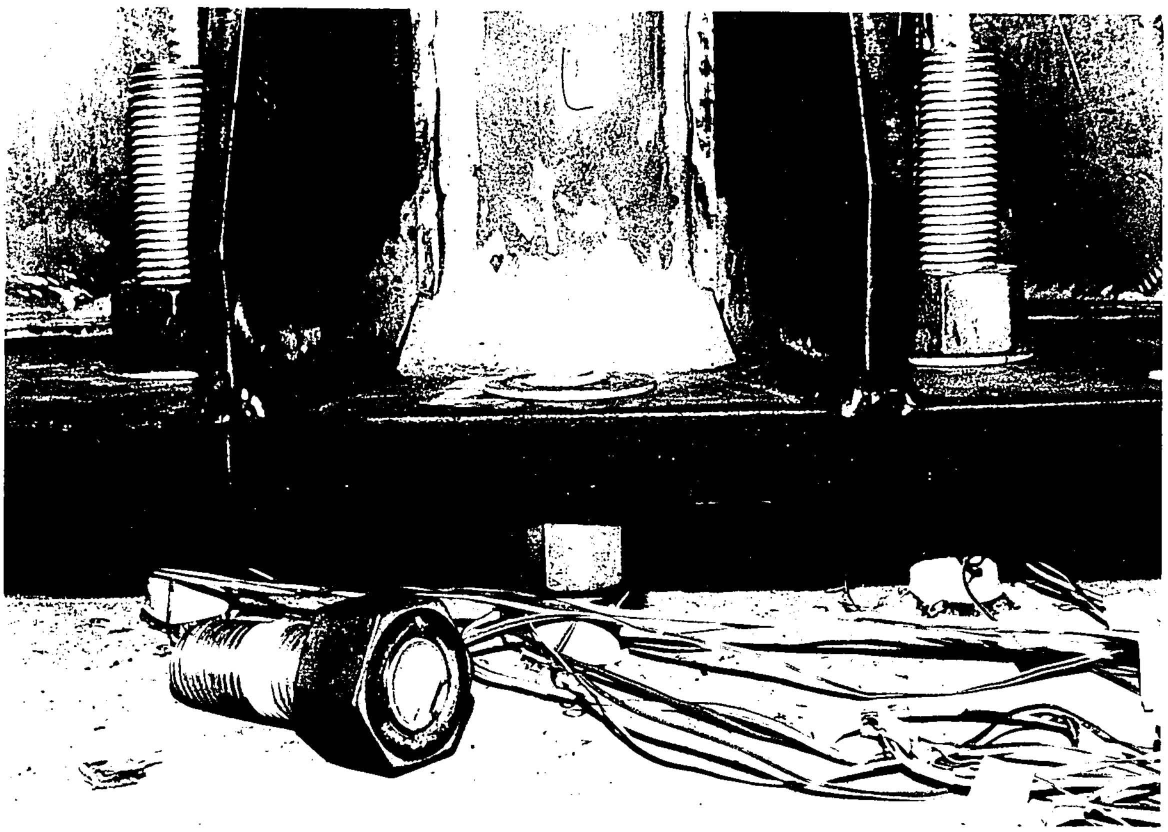


Figure 3.12 Closeup Photograph of Failure Location at Anchor Bolt

Chapter 4

Summary and Conclusions

4.1 Summary

Traditionally, cantilevered sign, signal, and luminaire support structures are designed using AASHTO's Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals without a significant number of problems. However, due to wind-induced vibrations of cantilevered support structures, some states have reported occurrences of fatigue damage. About 50% of these failures have occurred in the anchor bolts of these structures. The majority of which can be attributed to problems occurring during the installation process. These problems, as well as site-specific parameters, such as misalignment, exposed bolt length, and base plate thickness, affect the uncertainty in knowing the distribution of stresses in the anchor bolt assembly as well as the constant amplitude fatigue limit (CAFL).

Full-scale static tests were performed on two interchangeable specimens resulting in four assemblies, as described in Table 2.1. For each assembly tested, the actuator was placed to apply loads at four different locations, creating four different Moment:Torsion:Shear (M:T:V) ratios. Two ratios investigated the relationship of support-structure forces and anchor-bolt stresses which included torsion (19.5:5:1 and 14.5:10:1), while the other two did not (19.5:0:1 and

14.5:0:1). Additional tests that investigated the effects of looseness of nuts or failed bolts and the exposed bolt length above the concrete foundation were also performed for each assembly at at least one M:T:V ratio.

At the completion of the static tests, fatigue tests were then performed on the two concrete foundations using the column stub fabricated with the 1.5 in base plate. The actuator was situated at the M:T:V of 19.5:0:1 so that only horizontal shear and overturning moment were applied to the eight bolt group. A cyclic load was applied at a frequency of 1 Hz in only one direction so that a 20 ksi tension-tension stress range was measured in the critical bolt (bolt #1 or #5). The minimum was set to include the effect of the dead load, while the maximum stress was set at approximately 60 percent of the minimum yield stress. This maximum was to duplicate the conditions used in the individual anchor bolt fatigue tests. After the first bolt failed, the load process was reversed so that the other critical bolt could be tested similarly.

During the fatigue tests, the double-fillet welded socket connection between the column stub and the base plate failed under a 15 ksi stress range at 24,000 cycles. The detail was retrofitted with eight gusset stiffeners to meet the requirements of a Category C detail. The 0.5 in plate was ground and taper transitioned at a 15 degree angle from the wall of the column about 2 ft from the base plate. A full-penetration weld with fillet reinforcement was applied for 6 in near the termination of the stiffener.

Overall, the static tests were performed to confirm the assumptions made regarding the effect of the various parameters on the distribution of stresses in the bolts and to determine if using the flexure equation and the moment of inertia of the entire bolt group was appropriate to predict the distribution of stresses in the bolts. The fatigue tests were conducted to verify the uniaxial test results found through previous research, determining if these type of tests are a good representation of defining the fatigue strength of anchor bolts.

4.2 Conclusions

The results of an experimental study conducted to determine the relationship between support-structure forces and anchor-bolt stresses can best be defined by the effects the site-specific parameters (torsion, misalignment, looseness of nuts, base plate thickness, and exposed bolt length) had on the stress distribution of the bolts. Through a statistical study, it was determined that the presence of torsional loads affect the distribution of anchor bolt stresses most significantly.

Overall, the static tests on a full-scale, eight bolt anchor group proved that the distribution of axial anchor bolt forces can be reasonably predicted using the flexure formula, M*c/I with the moment of inertia of the bolt group. This relationship was found to be applicable for both straight and misaligned anchor bolt configurations as well as various Moment:Torsion:Shear ratios. By loosening one extreme bolt (bolt #1), the consequences of the failure of a bolt can be determined. The flexure equation was determined to be conservative when calculating the predicted increase in stresses in two bolts adjacent to the "failed" bolt.

Also, through tests conducted with the intentionally undersized 1 in thick base plate, prying effects were found to alter the distribution of stresses in the bolts. Prying is caused by non-uniforming bearing on the top or bottom nut as a result of base plate distortions. Therefore, the base plate must be sufficiently stiffened to limit these prying effects. One way to achieve this is to use a base plate with a thickness equal to or greater than one bolt diameter. The use of gusset stiffeners, such as those used to retrofit the specimen during the fatigue testing, will also help to reduce the prying effect.

Bending stresses caused by horizontal shear forces and torsional moments can be ignored in the design of anchor bolts when the exposed anchor bolt length (top of foundation to underside of leveling nut) does not exceed one bolt diameter. Typically, standard plans for cantilevered support structures specify that exposed lengths shall not exceed 1 in. However, for anchor bolt installations which do have exposed lengths greater than one bolt diameter, use of a fixed-fixed beam model is appropriate for calculating bending stresses that are additive to the axial stresses in the bolt.

Full-scale fatigue tests on the anchor bolt assembly provided results which were consistent with that for uniaxial tests. The CAFL corresponding to the AASHTO Category D design curve (7 ksi) is a reasonable lower-bound estimate for axially-loaded, snug- and fully-tightened anchor bolt. It was also determined that the presence of misalignment in the anchor bolt assembly does not affect the fatigue strength of the anchor bolts. Also, the double-fillet welded socket connection between the column stub and the base plate failed earlier than expected based on a Category E' prediction. However, when the detail was retrofitted with the addition of eight gusset stiffeners, no fatigue cracks were observed after the applications of approximately 1.4 million load cycles.

4.3 Future Research

Based on the findings of the research conducted during this project, the following suggestion is made for future research. Tests performed on the four assemblies, produced qualitative results regarding the relationship of anchor bolts stresses and column forces when subjected to several site-specific conditions (torsion, misalignment, base plate thickness, exposed bolt length, or looseness of nuts). However, if more tests are conducted to investigate the effect of these installation and site-specific parameters, a better quantitative characterization could be determined. Also, since the fatigue tests were conducted on the anchor bolt assemblies at a relatively high stress level, the effect of base plate prying was minimized. However, by conducting tests at loads closer to the fatigue limit, the proportion of the stress range due to base plate prying will be more significant and possibly influence the fatigue resistance.

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Vita

The author was born in Reading, PA, on September 11, 1972. She is the second of two children born to Mr. Miles K. and Mrs. Elizabeth Y. Dechant. She is engaged to be married to Luke E. Dewalt on October 19, 1996.

The author received her high school diploma from Governor Mifflin High School as salutatorian in June 1990. From there, she enrolled at Lehigh University where she obtained her Bachelor of Science in Civil Engineering in May, 1994. Directly after graduation, she was employed by Robert Swoyer Associates, an engineering consulting firm in Reading, PA. She also became certified as an Engineer-in-Training.

In the fall of 1994, she returned to Lehigh University to pursue her graduate work as a Master of Science candidate in the Department of Civil and Environmental Engineering with an emphasis in structures. During her graduate studies, she served as both a research assistant and a teaching assistant. Her area of research dealt with the fatigue resistant design of cantilevered sign, signal, and luminaire support structures. She specifically studied the relationship of support structure forces and anchor bolt stresses, which became the topic of her master's thesis. As a teaching assistant, she graded and taught recitation sections for Hydraulic Engineering and Measurements and Problem Solving in Civil Engineering.

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